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Nationwide Urban Runoff Program, USEPA (United States Environmental Protection Agency) Lansing, Michigan. Evaluation of Urban Stormwater Runoff and Management Practices for Controlling Urban Stormwater Runoff - Final Report

Tri-County Regional Planning Commission
Lansing, MI

Prepared for

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USEPA Nationwide Urban Runoff Program
Lansing, Michigan

EVALUATION OF URBAN STORMWATER RUNOFF
AND
MANAGEMENT PRACTICES
FOR CONTROLLING URBAN STORMWATER RUNOFF

For
Ingham County (Michigan) Drain Commission
and
Tri-County Regional Planning Commission
Lansing, Michigan

by
Environmental Control Technology Corporation

August, 1982
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SECTION I

ABSTRACT

An assessment of an on-line retention basin, two in-line upsize pipe reaches, and an off-line detention basin was undertaken as part of the USEPA Nationwide Urban Runoff Program. The study determined the effectiveness of the above mentioned treatment techniques in mitigating the pollutant impacts of urban runoff, as well as the operating costs associated with such installations.

The on-line retention basin receives runoff from 30.3 acres of mixed residential and commercial land use plus 36.7 acres of parkland. Twenty nine events were monitored with rainfall depths ranging from 0.14 to 1.02 inches.

The two in-line upsize pipe reaches are arranged in series within the existing storm drain system. The upstream pipe received runoff from 74.9 acres of mixed land use including residential, industrial and institutional. Eighteen events were monitored at the upstream upsize pipe, with rainfall amounts ranging from 0.22 to 1.63 inches. The downstream upsize pipe receives runoff from an additional 89 acres of mixed land use types for a total of 163.9 acres. The downstream land use mix includes residential, industrial, institutional, commercial, and parkland. Seventeen events were monitored at the downstream pipe, with rainfall amounts ranging from 0.34 to 0.96 inches.

The off-line detention basin drains 127.6 acres of mixed land use types, including residential, commercial, institutional, and parkland. The basin was monitored twice during the course of the study, once during an 0.86 inch rainfall and once during a 2.82 inch rainfall.

Concentrations of solids, nutrients, and metals were routinely monitored. The data obtained were used to determine water quality benefits, if any, derived from the treatment techniques, and land use/cover effects on storm runoff. Statistical analyses were used to determine relationships of pollutant concentration and load to total rainfall. In general it was found that parameter concentration was independent of rainfall and log-normal in distribution. Pollutant load was generally dependent on rainfall amounts.

The on-line retention basin was found to be very effective in treating a wide range of rainfall amounts and pollutant loads. The in-line upsize pipes displayed variable effectiveness depending on rainfall amount and runoff volume. The off-line detention basin had little or no effect on water quality.

SECTION II

INTRODUCTION

BACKGROUND OF STUDY

Section 208 of PL92-500 of the Clean Water Act, as administered by the U.S. Environmental Protection Agency, authorized and funded a planning process in most areas of the United States which was directed at evaluating the sources of water pollution and developing a management plan to correct and prevent water pollution. All sources of water pollution were considered - both point and non-point - and one conclusion from the overall effort was that urban non-point source runoff introduced a significant amount of pollution. Loadings of pollutants from urban runoff are capable of presenting significant shocks to the biological and chemical characteristics of the receiving waters. In addition to such pollutants as suspended solids, biochemical oxygen demand, fecal coliform and nutrients, urban runoff is capable of delivering other potentially more serious pollutants, such as heavy metals and toxic organic compounds. The latter can be introduced to the food chain and biomagnify. The ultimate impact of the pollutants in urban runoff may prevent the attainment of mandated quality goals and the beneficial uses associated with such goals.

The Tri-County Regional Planning Commission (TCRPC), Lansing, Michigan, as part of its overall 208 program, instituted a water quality monitoring program to document existing water quality in the Grand River and the other major streams in their jurisdiction. The results from this sampling program were used in the calibration and verification of the river water quality model used in the 208 planning process.

Using data collected during the 208 analysis as well as historical water quality information, the Tri-County Regional Planning Commission concluded that water quality problems do currently exist in the Grand River. These problems interfere with the full beneficial utilization of this water, particularly recreation. As is the case with the majority of urban rivers, these problems are caused by a combination of point source discharges, combined sewer overflows, and stormwater drainage. The final 208 Water Quality Management Plan indicated that non-point source pollution is the single largest contributor to biochemical oxygen demand, nitrogen, and suspended solids, and the control of this vector of pollution is thus a critical aspect with respect to achieving the desired water quality in the Grand River. The 208 plan went on to say that additional data will be needed to properly analyze the urban runoff problem in the Lansing metropolitan area.[1]

The Nationwide Urban Runoff Program is being undertaken by the USEPA in connection with the 208 Program, in order to assess the effectivenes and the cost of selected urban non-point source pollution prevention and correction techniques. The intent is to investigate and evaluate a wide range of alternative techniques which are termed, by USEPA, Best Management Practices (BMP's). Evaluating BMP's nationwide was the the policy option selected by the USEPA as the most prudent method of developing the body of information needed to correct and prevent pollution from urban stormwater runoff. The 208 Program resulted in the collection of much water quality data, but very little information exists, or has been collected by 208 studies heretofore, on the cost and effectiveness of water pollution control techniques. The NURP is supporting the evaluation of selected BMP's throughout the country, and, based on such evaluations, will determine which BMP's are cost-effective in a particular situation and at a given scale. This approach will yield data which is comparable within and between geographic regions. A collection of information will be available at the conclusion of NURP which will provide guidance to Federal, regional, State and local agencies responsible for implementing the Areawide Water Quality Management Plans.

With this explanation of the evolution of the Nationwide Urban Runoff Program in mind, the objectives of the project conducted in Lansing, Michigan are hereby presented.

OBJECTIVES OF THE LANSING, MICHIGAN NURP

The major objective of this project was a thorough evaluation of the three types of "best management practices"

(BMP's) incorporated within the Bogus Swamp Drain Drainage District. The BMP's evaluated include an in-line wet retention basin, an off-line dry detention basin, and two up-sized pipe sections. This study was designed to determine the efficiency of these practices in the enhancement of storm water quality and to determine all associated costs, both capital and operating, for each of the management practices.

Several sub-objectives were also accomplished during this project in order to meet the primary objective. The pollutant loads in the stormwater entering and leaving each BMP were determined, and relevant information concerning land use, runoff and pollutant levels were obtained. Annual loads of various pollutants which are derived from specific sub-basins within the district, as well as the discharge from the entire district, were determined.

Another objective in thoroughly evaluating the BMP's was an assesment of the impact these practices could have on the receiving water quality (Grand River) both directly in the Bogus Swamp Drain area and on a region wide basis, should similar BMP's be installed in other drainage basins.

An additional activity was the delineation of the cost of the application of these BMP's to this type of urban drainage system. The determination of these costs, along with the evaluation of the effectiveness of BMP's in enhancing drainage water quality, allows for the evaluation of the cost effectiveness of such practices.

The final objective was and is the dissemination of the information obtained in this study throughout the region, allowing for its application in other drainage basins.

EXISTING INFORMATION

Substantial amounts of historical information are available with respect to the Grand River. Water quality monitoring has been routinely performed by the Michigan Department of Natural Resources (MDNR) at numerous stations for many years. In addition, the Grand River Watershed Council. with the assistance of local wastewater treatment plant personnel, have monitored additional stations. As of 1977, there were seven stations on the Grand River within the Tri-County region which were being monitored on a monthly basis. All of the data generated from these monitoring programs are recorded in the STORET system.

Point source pollution inputs to the river system are also well documented. In addition to the monthly operating reports submitted to the MDNR by each discharger, Michigan's Bureau of Water Management (MBWM) surveyed essentially all industrial and municipal point source dischargers in the decade prior to the establishment of final effluent limitations in 1977. In addition, the MBWM performed comprehensive river monitoring programs, biological surveys, and bioassays for various river reaches.

Monitoring programs such as those discussed above serve to provide an indication of long-term trends in water quality. In general, however, they do not provide an indication of the impacts of intermittent sources of pollution, such as combined sewer overflows (CSO's), urban storm sewers, and open storm drains. Very little study of these intermittent pollution sources had been performed in the region prior to the inception of the 208 Water Quality Management Program.

During 1976, as a part of their 208 program, the TCRPC monitored urban stormwater runoff at seven stations, representing four types of land usage. Based on the results of this monitoring program, along with data obtained from the programs listed above, they were able to estimate the relative significances of urban non-point runoff with respect to the quality of the receiving water. It was projected that, during summer storm events, non-point sources accounted for 25% of the BOD loading, 21% of the total Kjeldahl nitrogen loading, 34% of the suspended solids loading, and 10% of the total phosphorus loading to the Grand River. On a total annual loading basis, the estimated contribution from non-point sources was lower, but still significant, providing 19% of the BOD, 13% of the TKN, 27% of the suspended solids, and 5% of the phosphorus.[2]

INITIAL ASSUMPTIONS

Based on this background information the following assumptions were made in designing and implementing this study:

1. Urban runoff contains significant quantities of pollutants and a major fraction of these pollutants are in the suspended form.

2. Best Management Practices which provide for the settling of suspended material would be effective in reducing pollutant loads.

3. Stormwater characteristics are highly variable and difficult to predict. However, the basic theory of sedimentation should apply, and thus factors such as surface area, detention time, velocity, particle size, etc. should provide a basis for evaluating removal mechanisms.

4. Retention/detention basins provide the types of conditions which would allow sedimentation to occur and thus would be appropriate Best Management Practices.

5. BMP's of this type, assuming they were effective, were practical for "real world" use in controlling urban runoff pollutants, particularly as they were currently being designed and constructed for flood control purposes.

6. Implementation of BMP's would result in reduced pollutant loads to the receiving stream with subsequent improvement in water quality conditions.

SECTION III

DESCRIPTION OF STUDY AREA

GRAND RIVER BASIN

General

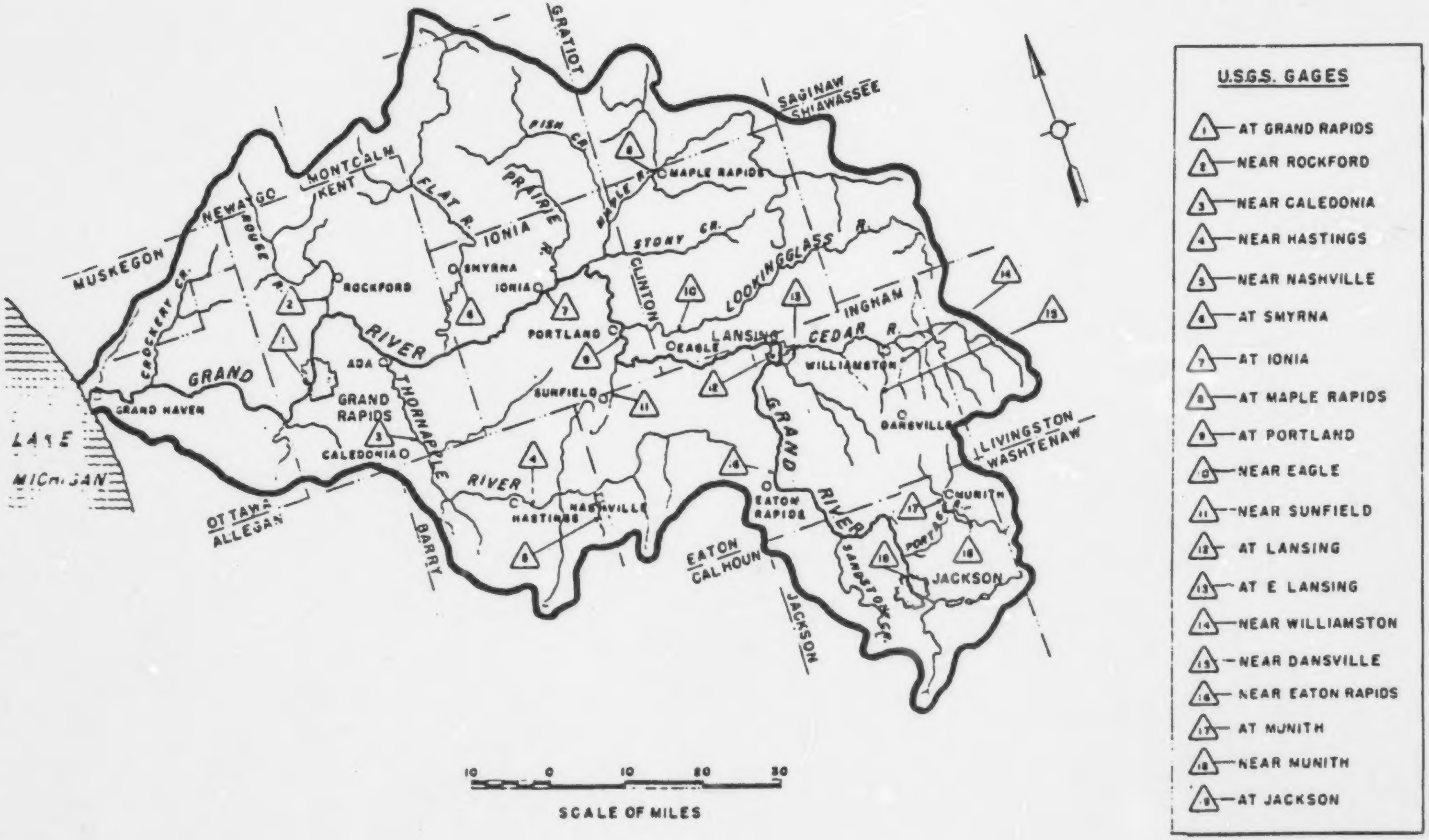
The Grand River Basin (Figure III-1) with a drainage area of approximately 5,622 square miles lies in the southwestern portion of lower Michigan and is comprised of parts of Washtenaw, Jackson, Livingston, Ingham, Eaton, Barry, Sniawassee, Clinton, Gratiot, Ionia, Montcalm, Kent, Newaygo, Ottawa, and Muskegon Counties. The major cities in the basin include Jackson, Lansing, and Grand Rapids. The basin length is approximately 125 miles, and the width is approximately 80 miles at the widest point. It is bounded on the north by the Muskegon River Basin, on the northeast by the Saginaw River Basin, and on the south by the Kalamazoo River Basin. Major tributaries include the Red Cedar, Lookingglass, Maple, Prairie, Flat, Rogue and Thornapple Rivers and Crockery Creek.

The Grand River flows north from its headwaters in Jackson County, through Eaton County, and into Lansing in Ingham County where it is joined by the Red Cedar River. The Red Cedar River begins in Cedar Lake in Livingston County and flows in a westerly direction along a course of roughly 50 miles to its confluence with the Grand River. The total drainage area of the Grand River at Lansing is approximately 1230 square miles, with 355 square miles of this total area being contributed by the Red Cedar.

21

FIGURE III-1

Grand River Basin


U.S.G.S. GAGES
1 AT GRAND RAPIDS
2 NEAR ROCKFORD
3 NEAR CALEDONIA
4 NEAR HASTINGS
5 NEAR NASHVILLE
6 AT SMYRNA
7 AT IONIA
8 AT MAPLE RAPIDS
9 AT PORTLAND
10 NEAR EAGLE
11 NEAR SUNFIELD
12 AT LANSING
13 AT E LANSING
14 NEAR WILLIAMSTON
15 NEAR DANSVILLE
16 NEAR EATON RAPIDS
17 AT MUNITH
18 NEAR MUNITH
19 AT JACKSON
LAKE MICHIGAN
GRAND HAVEN
GRAND
CROCKERY CR.
MUSKEGON
NEWAYGO
MONTCALM
KENT
ROUGE R.
ROCKFORD
GRAND RAPIDS
ADA
CALEDONIA
OTTAWA
ALLEGAN
BARRY
THORNAPPLE
RIVER
FLAT R.
IONIA
SMYRNA
PRAIRIE R.
FISH CR.
GRATIOT
MAPLE R.
MAPLE RAPIDS
STONY CR.
CLINTON
PORTLAND
SUNFIELD
HASTINGS
NASHVILLE
EAGLE
LANSING
LOOKINGGLASS R.
SAGINAW
SHIAWASSEE
INGHAM
CEDAR R.
WILLIAMSTON
DANSVILLE
LIVINGSTON
WASHTENAW
GRAND RIVER
EATON RAPIDS
EATON
CALHOUN
JACKSON
SANDSTONE CR.
PORTAGE R.
MUNITH
10 0 10 20 30
SCALE OF MILES

The mean flow of the Grand River at Lansing is 824 cfs, and the mean flow of the Red Cedar at East Lansing is 201 cfs. There are five impoundments affecting the flow of the Grand River in this area. Three are in the Eaton Rapids area (approximately 25 river miles upstream of Lansing) and two are in the City of Lansing. Flows of the Red Cedar are influenced by dams located in East Lansing and at Williamston (approximately 15 miles east of Lansing). Large diurnal fluctuations at medium and low flows have resulted from activities at the power plants above the Lansing gauging station.

Topography

The land surface of the Grand River Basin around Lansing varies from generally flat to a gently rolling contour. The highest elevation in the region, 1,052 feet, occurs in southern Ingham County. The lowest elevation 640 feet, occurs at the northwestern boundary of Clinton County. The major land surface features were formed during the Wisconsin glacial epoch about 15,000 years ago. Much of the land is till plain, formed by stagnant or constantly receding ice sheets. Glacial drainage channels, now broad swampy valleys and glacial depressions, consisting of muck and peat swamps, are common throughout the region. Roughly 80-90% of the region is flat, with less than 5% slopes. Few areas have slopes greater than 10%, these areas are usually small relief components such as eskers, recessional moraines, and streambanks.

Geology

The geological composition of the area consists of unconsolidated glacial derived clay, silt, sand, and gravel overlying consolidated sediments of approximately 10,000 feet in depth. The consolidated sediments which now comprise the bedrock formations consist of layers of sandstone, limestone, and shale which were deposited in ancient seas 200 to 300 million years ago during the Paleozoic Era. After the ancient seas had disappeared the consolidated sediment became exposed to the forces of weathering and erosion for approximately 200 million years extending through most of the Mesozoic and Cenozoic Eras. A series of four glacial events created the unconsolidated topographic features and glacial aquifers which now exist in the region. The "ice age" or Pleistocene period began one million years ago and subsided about 15,000 years ago.

The surface of the bedrock is encountered at depths ranging from about 50 to 200 feet. The bedrock formations are composed primarily of sandstone, limestone, dolamite, and shale. These rock formations are the principal sources for drinking water supplies in the region. The rock strata below a depth of 500 feet may contain saline water.

A "loamy" glacial till material was deposited over most of the region and evolved into the fertile soils of today. Much of the soil in the region can be classified in a medium to high fertility range. Although the fertile clay loam and sandy loam soils are found over much of the region, loamy sand, sand, and organic soils predominate in some areas.

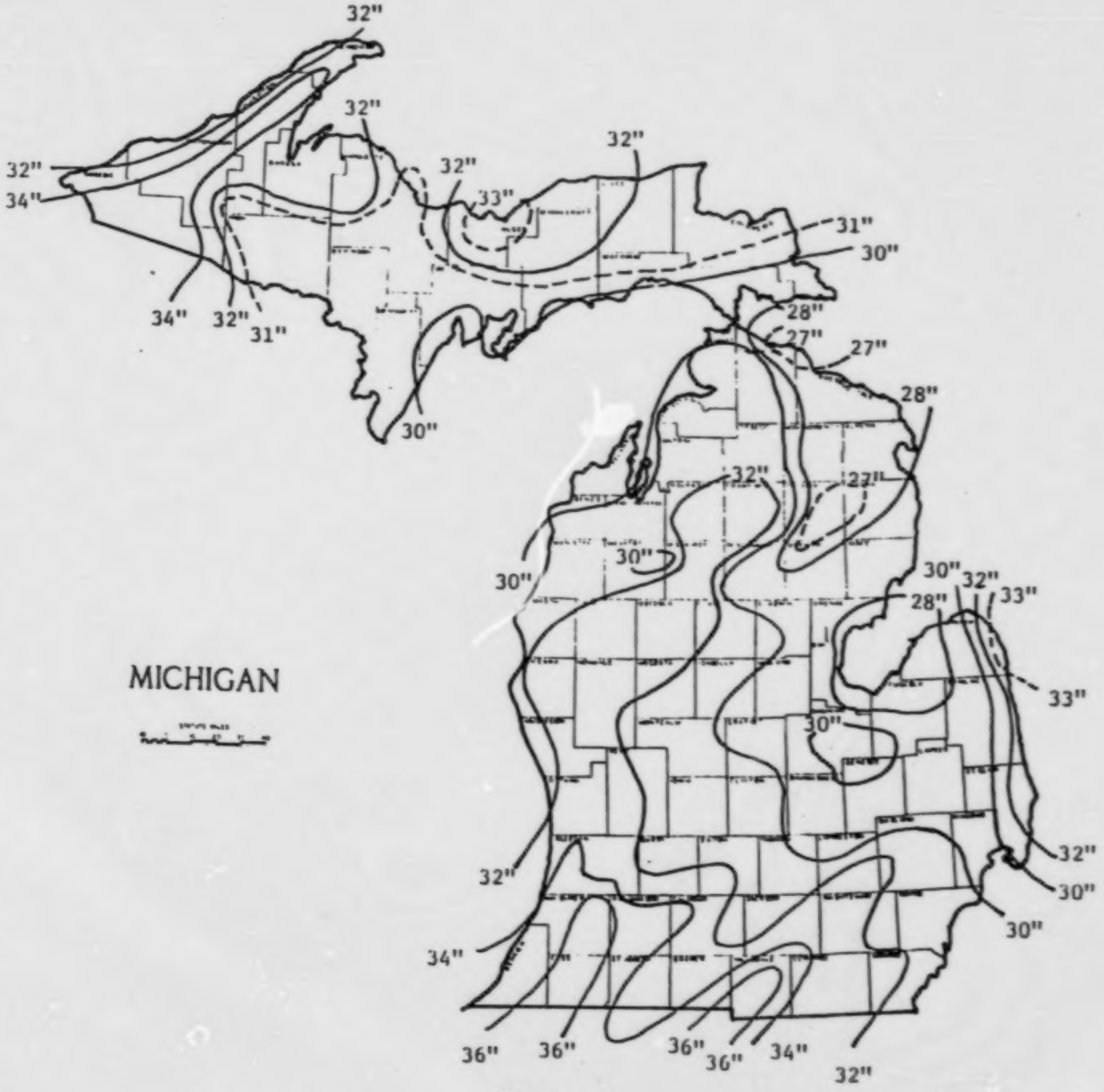
Climate

The Lansing area experiences a climate alternating between continental and semimarine, depending on meteorological conditions. The marine type is governed by the force and direction of the wind and is influenced by the Great Lakes. Continental weather results from little or no wind movement, which creates a pronounced fluctuation in temperatures, including hot weather in the summer and severe cold in the winter. A strong wind from the Great Lakes may immediately transform the weather into a semimarine type. Because of the region's midlatitude location, prevailing winds are westerly. During the summer months winds are predominantly southwesterly as the Bermuda High Pressure Center pushes into the southeastern United States. During the winter months the prevailing winds are west to northwest, but change quite frequently for short periods as migrating cyclones and anti-cyclones move through the Midwest.

Mean temperatures range from a high of 70.7°F in July to a low ot 22.2 °F in January. The mean annual volume of precipitation for the period 1940-1969 was approximately 30 inches, as shown in Figure III-2. Monthly minimum rainfall is generally 1.3 to 1.6 inches and maximum monthly rainfall ranges from 3.5 to 4.2 inches. Annual snowfalls range from 37 to 51 inches.

FIGURE III-2

ANNUAL MEAN PRECIPITATION (inches), 1940-1969


32"
32"
32"
34"
32"
33"
31"
30"
34"
32"
31"
30"
28"
27"
27"
28"
32"
27"
30"
30"
30"
32"
33"
28"
33"
MICHIGAN
30"
32"
32"
30"
30"
34"
36"
36"
36"
36"
34"
32"

LANSING AREA - LEVEL I

General

The Lansing area depicted in Figure III-3 encompasses nine townships including and surrounding the cities of Lansing and East Lansing. The Lansing urban area is also outlined in Figure III-3. The outlined area represents areas presently serviced by sanitary sewers.

The topography of the Lansing area is relatively flat with few areas (Figure III-4) exceeding slopes of 10 percent. Many of the slopes which exceed 5 percent are found contiguous with outwash areas identified in Figure III-5. The areas which have slopes of less than 5 percent are usually associated with the till moraine areas also identified in Figure III-5.

Drainage Systems

The major surface drains and watershed boundaries in the Lansing area are defined in Figure III-6. Figure III-6 also identifies stream dam locations and USGS monitoring stations.

The major surface drains, including the Grand, Red Cedar and Looking Glass Rivers, are old glacial drainage channels. The channels were formed when the last

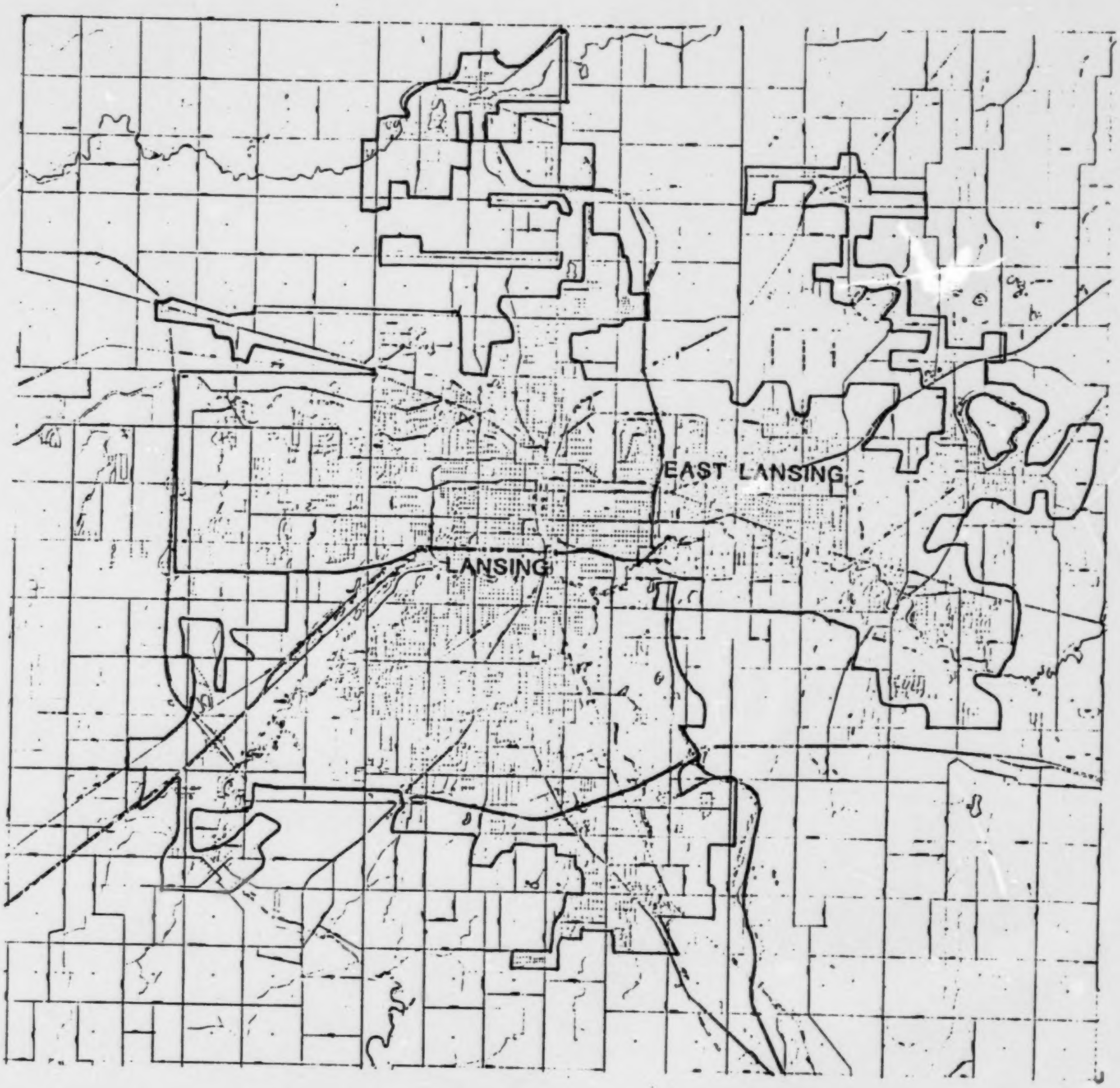
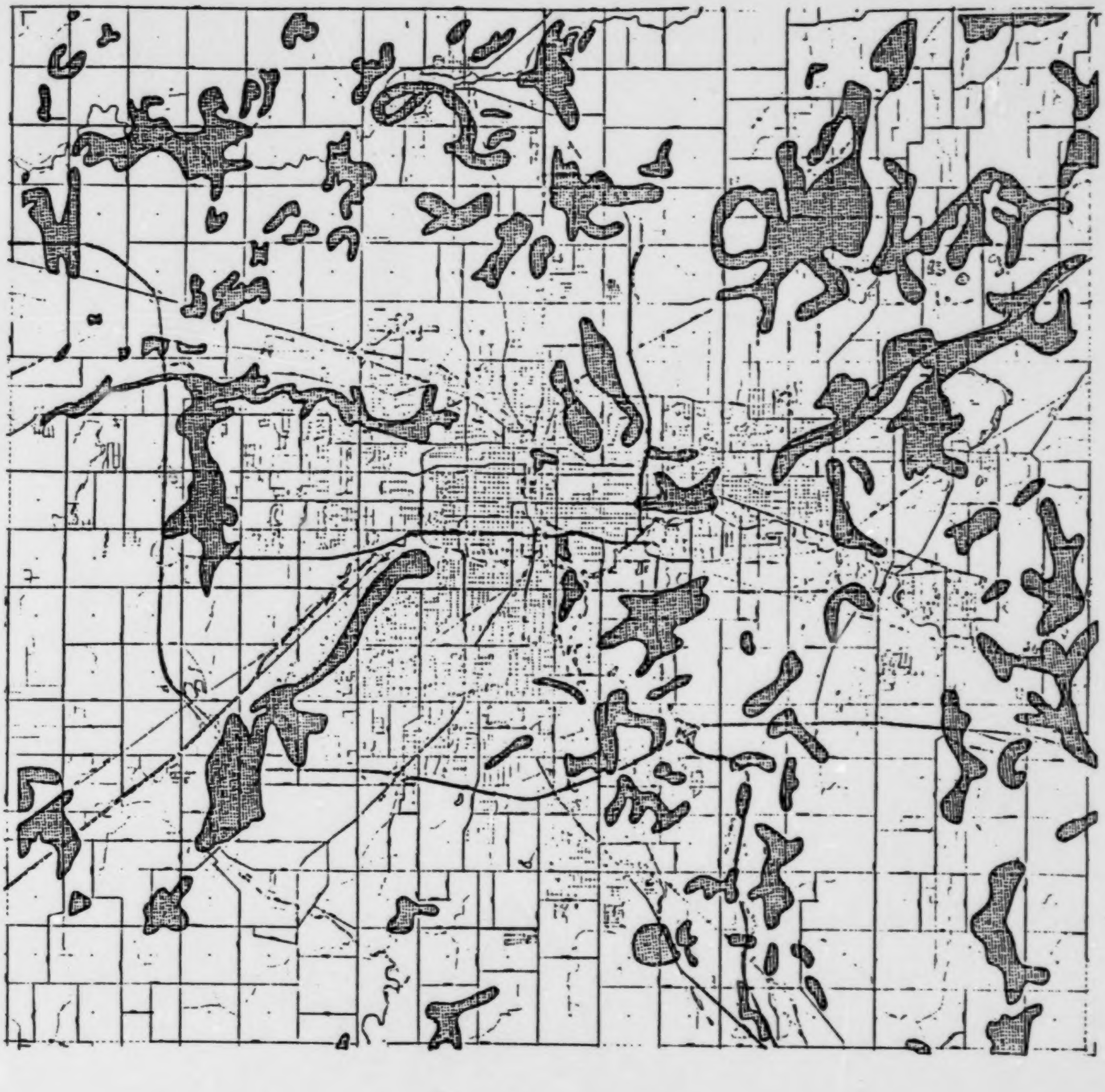

EAST LANSING
LANSING

Figure III - 3 Level I Lansing Urban Area



▨ **Slope Areas Greater Than 5 Percent**

Figure III - 4 Areas With Slope Greater Than 5 Percent

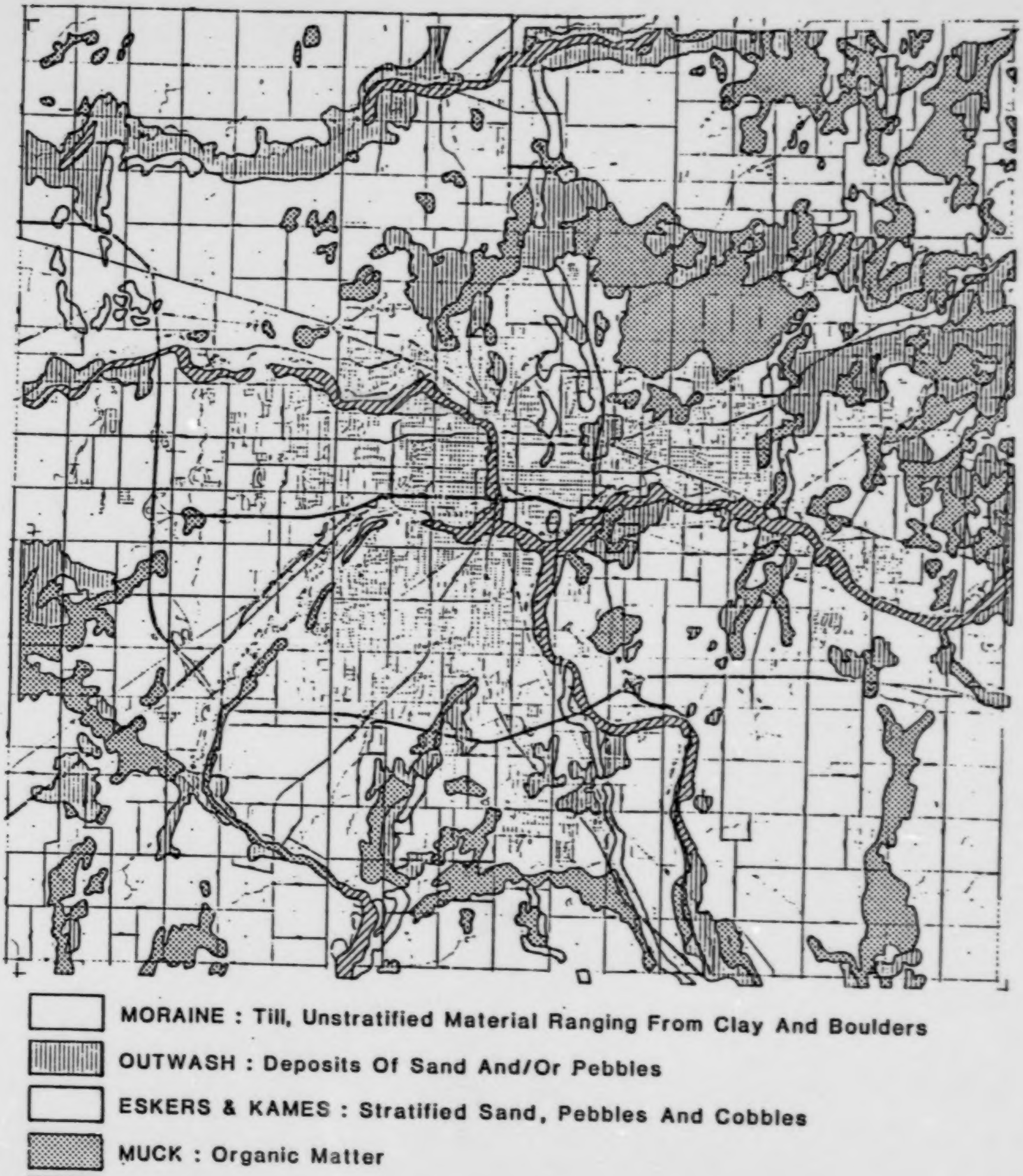

MORAINE : Till, Unstratified Material Ranging From Clay And Boulders
OUTWASH : Deposits Of Sand And/Or Pebbles
ESKERS & KAMES : Stratified Sand, Pebbles And Cobbles
MUCK : Organic Matter
ALLUVIUM : Silt And Sand Deposited On Flood Plains

Figure III - 5 Surface Geology

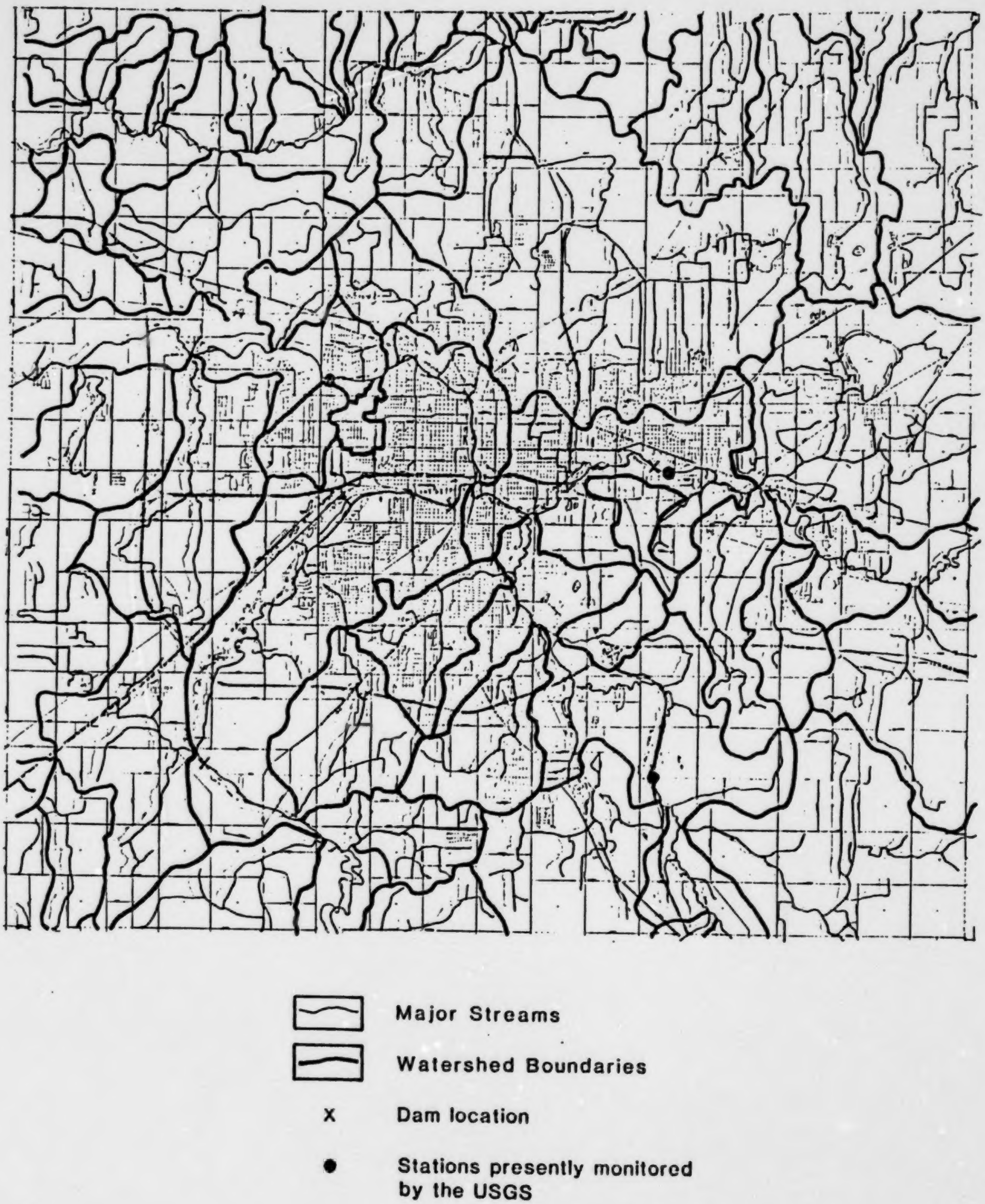

Major Streams
Watershed Boundaries
X Dam location
● Stations presently monitored by the USGS

Figure 111 -6 Drainage Basins

continental glacier melted as it remained in a stationary or receding conditions over 10,000 years ago. At the same time, glacial debris from the melting ice was deposited along the northern side of the channels and, thus, created many of the recessional moraines in the area.

Soils

The comprehensive soil survey reports for Clinton, Eaton, and Ingham counties were utilized to identify soils with medium to rapid seepage rates. The seepage rate was defined as the rate at which water will percolate and move laterally and vertically through the soil profile. The soils found within the region which have medium and/or high seepage rates and relatively high permeability rates are listed in Table III-I and identified in Figure III-7. All of the soils identified in Figure III-7 have gravel, sand, loamy sand or sandy loam textures. The seepage rate criteria is often used by planners to help evaluate whether soils are suitabile for the accommodation of waste disposal systems artificial or natural recharge facilities and various types of impoundments. Soils with medium of high seepage rates may have serious problems if their intended use is for private septic system effluent filter fields, landfills or effluent lagoons. The soils with medium to high seepage rates will generally have medium to high infiltration and transmission rates.

TABLE III-I

Characteristics of Soils Identified in Figure III-7

Soil Series	Map Symbol	Soil Mgt. Gp.	Seepage Rate	Permeability (In per hr.)
Boyer	Bn	4a	med-rapid	2-20
Boyer	Bo	4a	med-rapid	2-20
Boyer	Br	4a	med-rapid	2-20
Boyer-Spinks	Bs	4a	med-rapid	2-20
Ceresco	Ce	L-2C	med	0.6-6
Cohoctah	Ch	L-2C	med	0.6-6
Granby	Gr	5c	rapid	6-20
Spinks-Metea	St	5c	rapid	6-20
Thetford	Th	4b	med-rapid	2-20
Boyer-Spinks	Ue		med-rapid	2-20
Washtenaw	Wd	L-2c	med	0.6-2
Oshtemo	Ot	4a	med-rapid	2-20
Oshtemo	Os	4a	med-rapid	2-20

Information assembled from the soil surveys of Ingham County, 1979; Eaton County, 1978; and Clinton County, 1979.

33

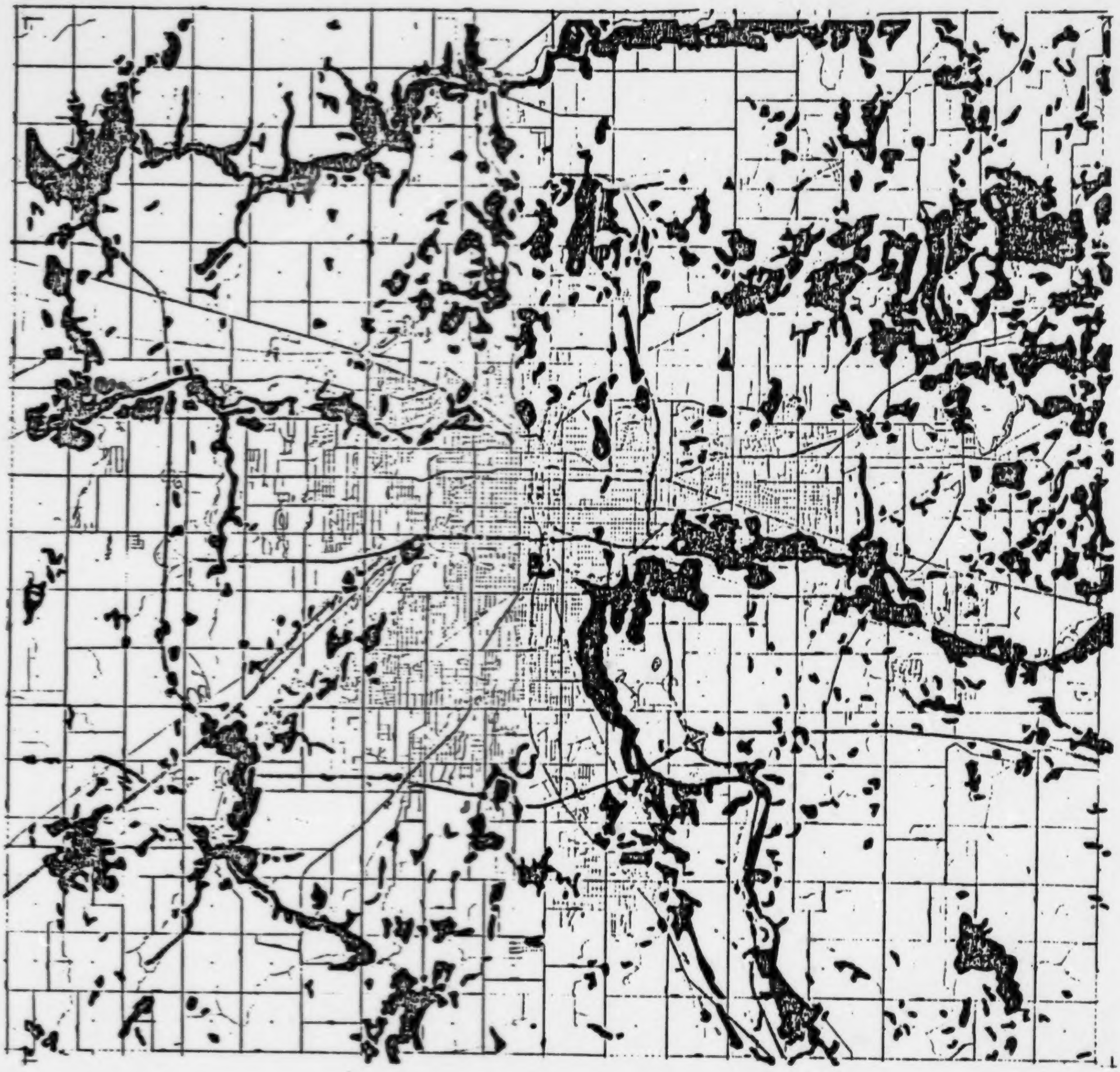


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Figure 111 - 7 Permeable Soils

Land Use

The land use types and totals for the Lansing area are summarized in Table III-2. As can be seen by referring to Table III-2, 37% of the area in the nine township Lansing area is agriculture land. Urbanized land occupies 21% of the area. forest lands 9% and water-wetlands the remaining 5%. The urbanized land, as depicted in Figure III-3, consists of general areas which presently have sanitary sewers.

TABLE III-2

Land Use Designations and Averages For the Lansing Area - Level I

Category		Acres	Percent
1000	Urban	39,242	21
2000	Agriculture	122,335	65
3000	Rangeland	-0-	-0-
4000	Forest	17.122	9
5000	Water	2,610	1
6000	Wetlands	6.998	4
7000	Barren	-0-	-0-
8000	Other	-0-	-0-
	TOTAL	188,307	100

36

LANSING METROPOLITAN AREA - LEVEL 2

General

The Lansing metropolitan area includes the cities of Lansing, East Lansing, Haslett, Holt, Okemos and Dimondale as depicted in Figure III-8. Generally, the topography of the Lansing metropolitan is relatively flat with a few sloping areas associated with outwash, alluvium and moranic areas depicted in Figures III-4 and III-5.

Drainage Systems

The major surface drains in the area include the Grand River and its tributaries the Red Cedar River and Sycamore Creek. The surface drainage system is depicted on Figure III-9.

Soils

Nine major soil associations have been identified by the Soil Conservation Service, United States Department of Agriculture, within the Lansing metropolitan area. The soil associations are depicted in Figure III-10 and discussed according to drainage characteristics in Table III-3.

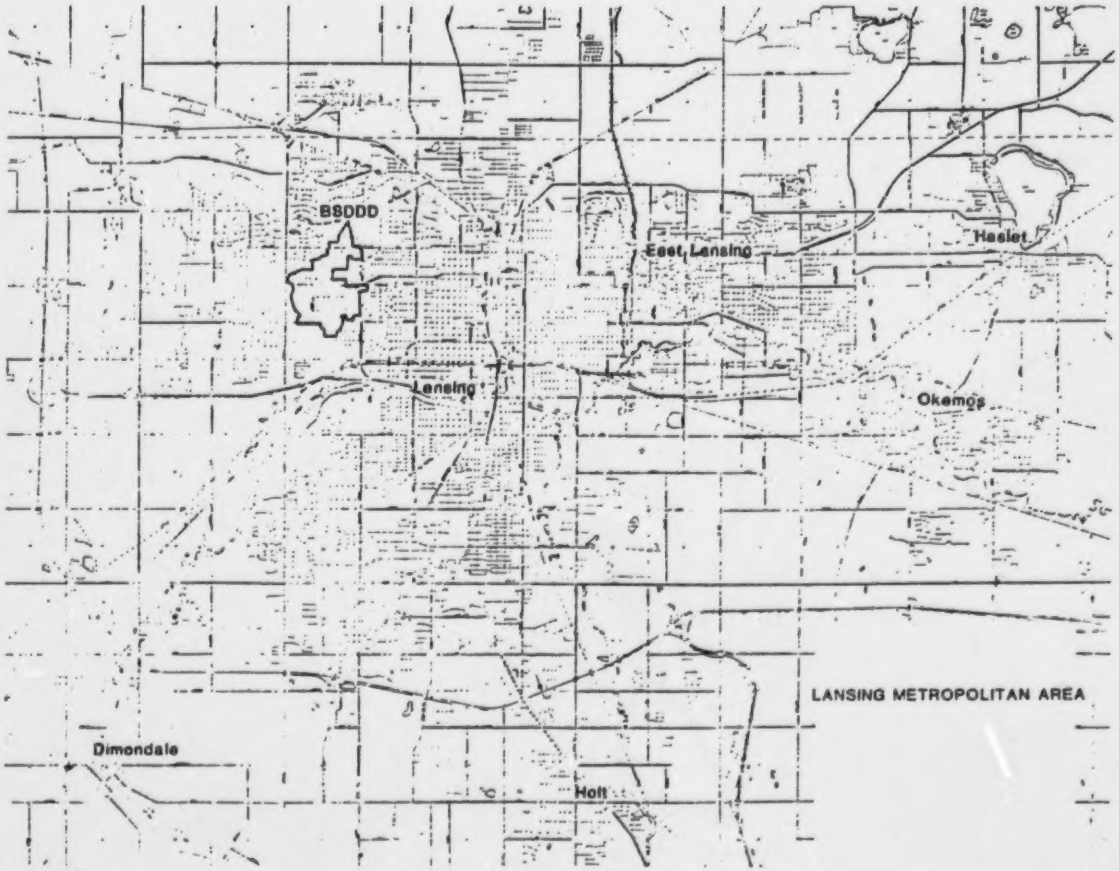
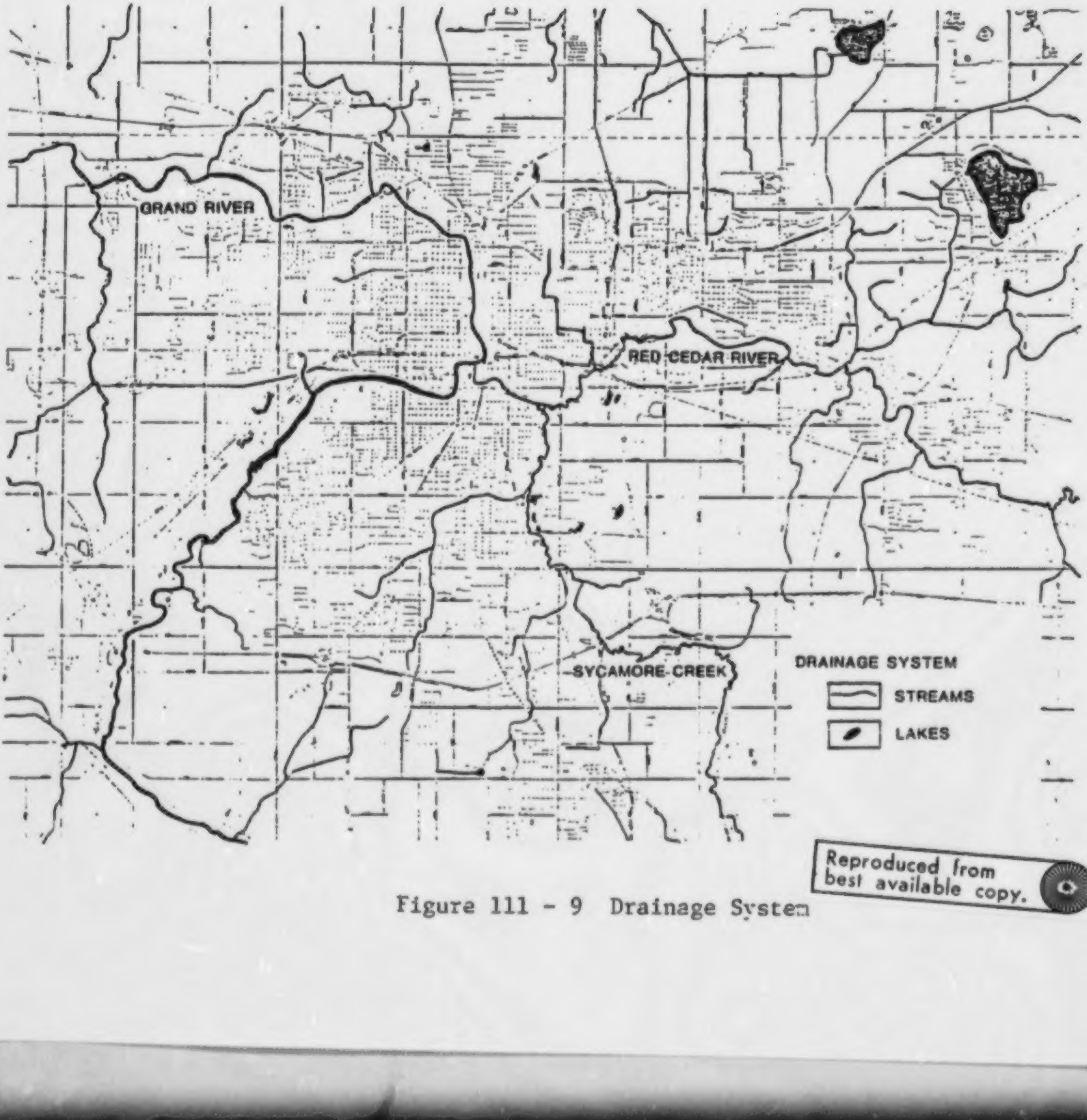

BSODD
East Lansing
Haslet
Lansing
Okemos
LANSING METROPOLITAN AREA
Dimondale
Holt

Figure III - 8 Study Area Location


GRAND RIVER
RED CEDAR RIVER
SYCAMORE CREEK
DRAINAGE SYSTEM
STREAMS
LAKES

Reproduced from best available copy.

Figure III - 9 Drainage System

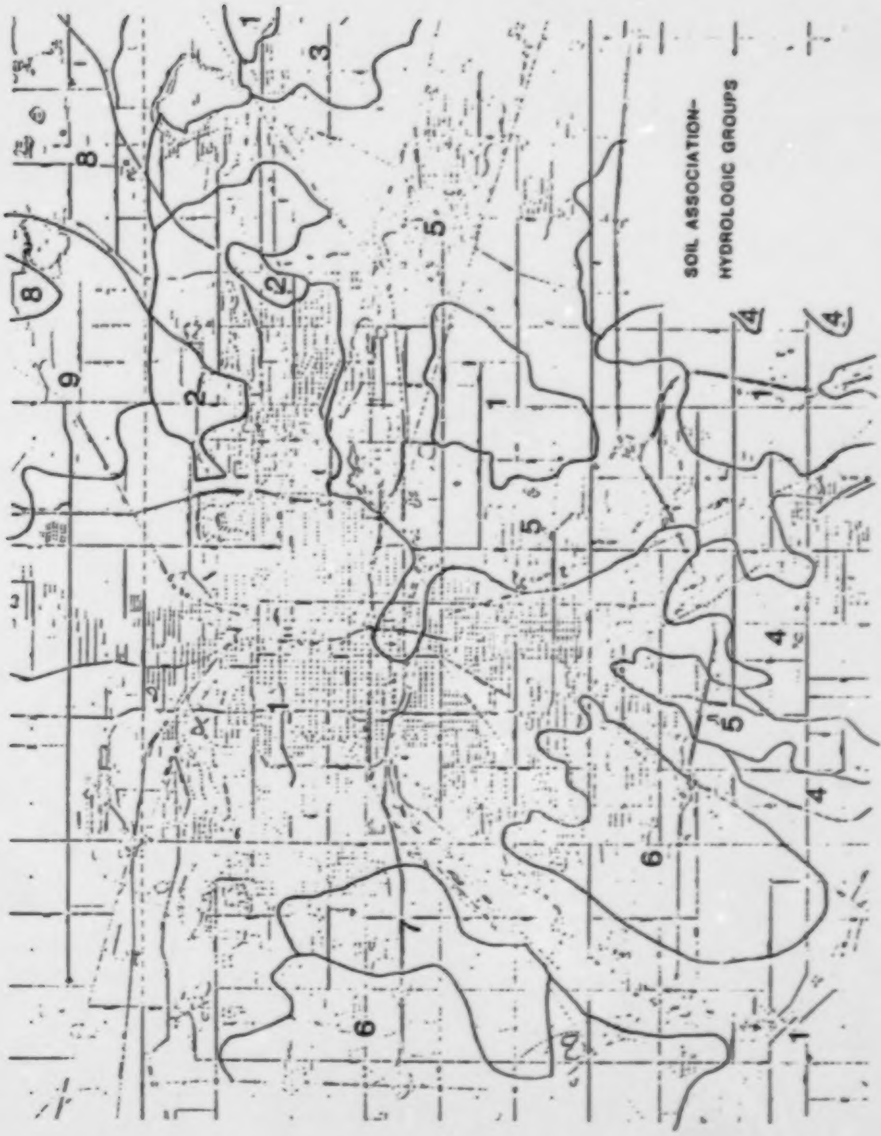

SOIL ASSOCIATION-
HYDROLOGIC GROUPS
1
2
3
4
5
6
7
8
9

Figure III - 10 Soil Types

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best available copy.

TABLE III- 3

Legend for Soils Identified in Figure III-10

Identification Number	Soil Association Name	Characteristics and Locations
1	Marlette-Capac-Owosso-Parkhill-Association	Nearly level to sloping. drained sandy loams and nearly level somewhat poorly drained loams. Small areas of Riddles and Hillsdale sandy loams and Brookston Loam are included. (Moderately high runoff potential)
2	Houghton-Palms-Edwards Association	nearly level, poorly and very poorly drained soils. Small areas of Adrian muck and Sebewa loam are included. (Low runoff potential)
3	Oshtemo-Houghton-Riddles Association	nearly level to steep. well drained loamy sands and sandly loams and nearly level, poorly and very poorly drained organic soils. Also included are small areas of Aubbeenaubbee and Gilford sandy loams. (Moderately low runoff potential)
4	Capac-Brookston-Marlette-	nearly level, somewhat poorly and poorly drained loams and moderately well or well drained sandy loams. Small areas of Oshtemo loamy sand and Aubbeenaubbee sandy loam are included (Moderately low runoff potential)

TABLE III-3 Continued

Legend for Soils Identified in Figure III-10

Identification Number	Soil Association Name	Characteristics and Locations
5	Marlette-Oshtemo-Capac Association	nearly level to steep. drained sandy loams and loamy sands and nearly level, somewhat poorly drained loams. Small areas of Brady sandy loam. Houghton muck and Brookston loam are included. (Moderately high runoff potential)
6	Marlette-Capac Association	nearly level to gently undulating, well drained to somewhat poorly drained loamy soils on til plains. (High runoff potential)
7	Capac-Parkhill Association	nearly level to very gently sloping, somewhat poorly drained and nearly level poorly and very poorly drained loamy soils on til plains and low moraines. (Moderately high runoff potential)
8	Boyer-Marlett-Houghton Association	well and moderately well drained, gently sloping to steep loamy sand, sandy loam, and loam soils on moraines and very poorly drained muck soils in depressions. (moderately low runoff potential)

TABLE III-3 Continued

Legend for Soils Identified in Figure III-10

Identification Number	Soil Association Name	Characteristics and Locations
9	Houghton-Gilford	very poorly drained. nearly Adrain Association level muck and sandy loam soils on old lake basins and depressed drainageways. (Low runoff potential)

Land Use

The land use types illustrated in Figures III-11 through III-13 include urban residential, commercial and industrial. Figure III-14 depicts forested areas within the Lansing metropolitan area. The acreages for the land use types are summarized in Table III-4.

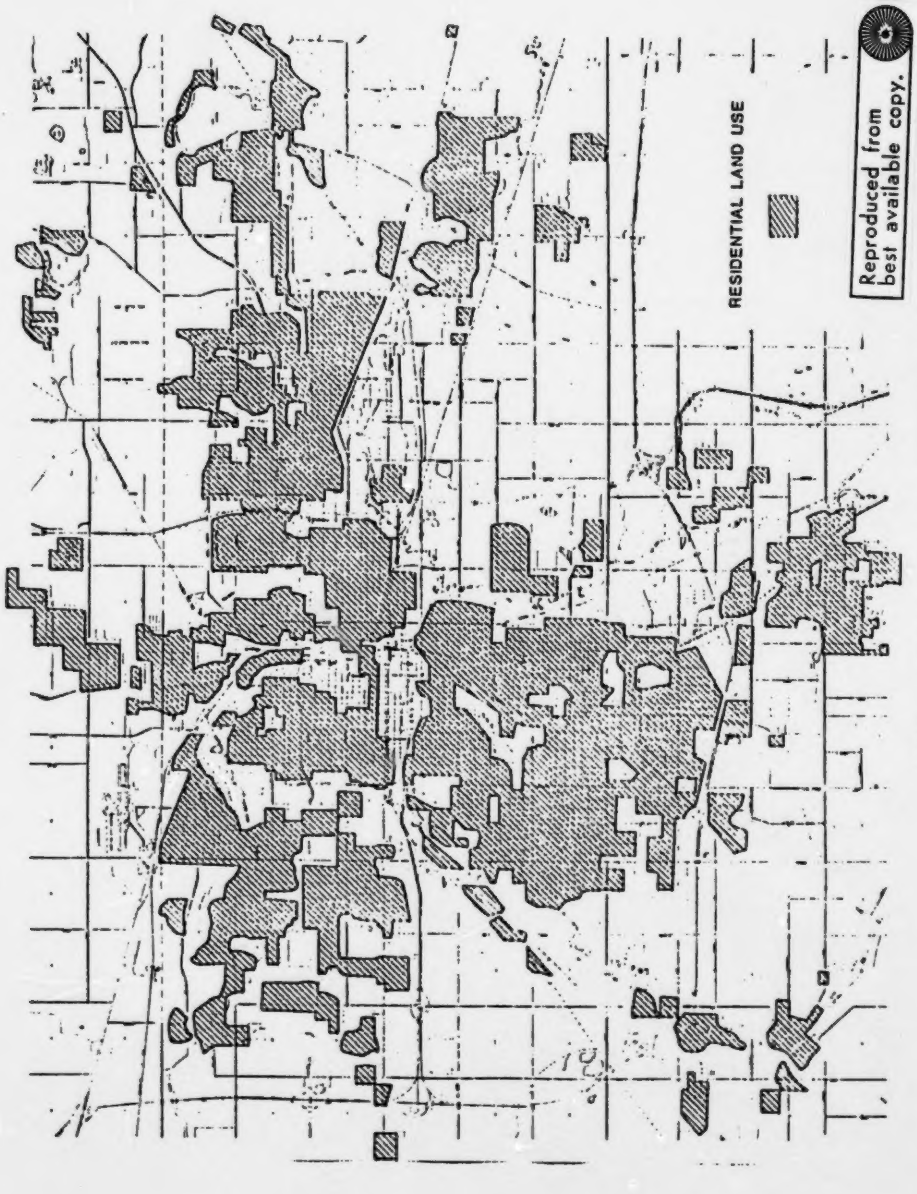
RESIDENTIAL LAND USE
▨

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Figure III - 11 Land Use - Residential

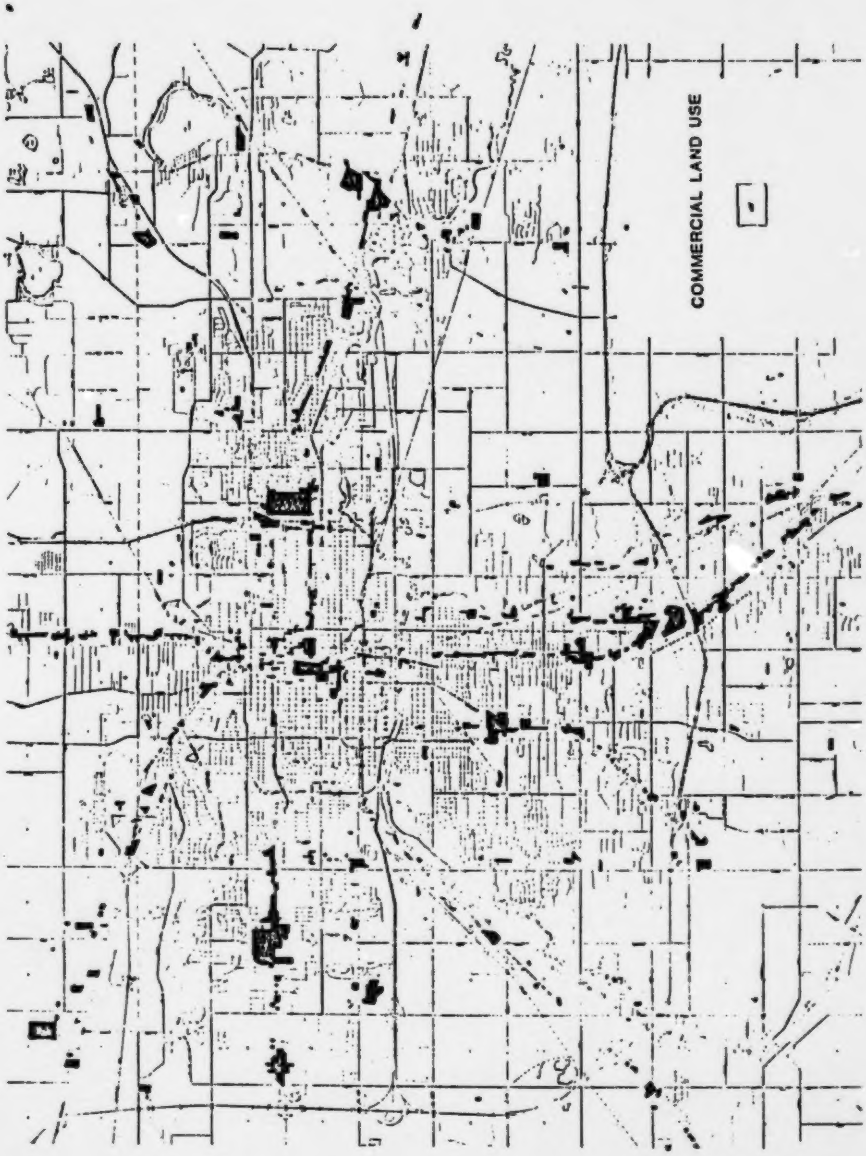
COMMERCIAL LAND USE

Figure 111 - 12 Land Use - Commercial

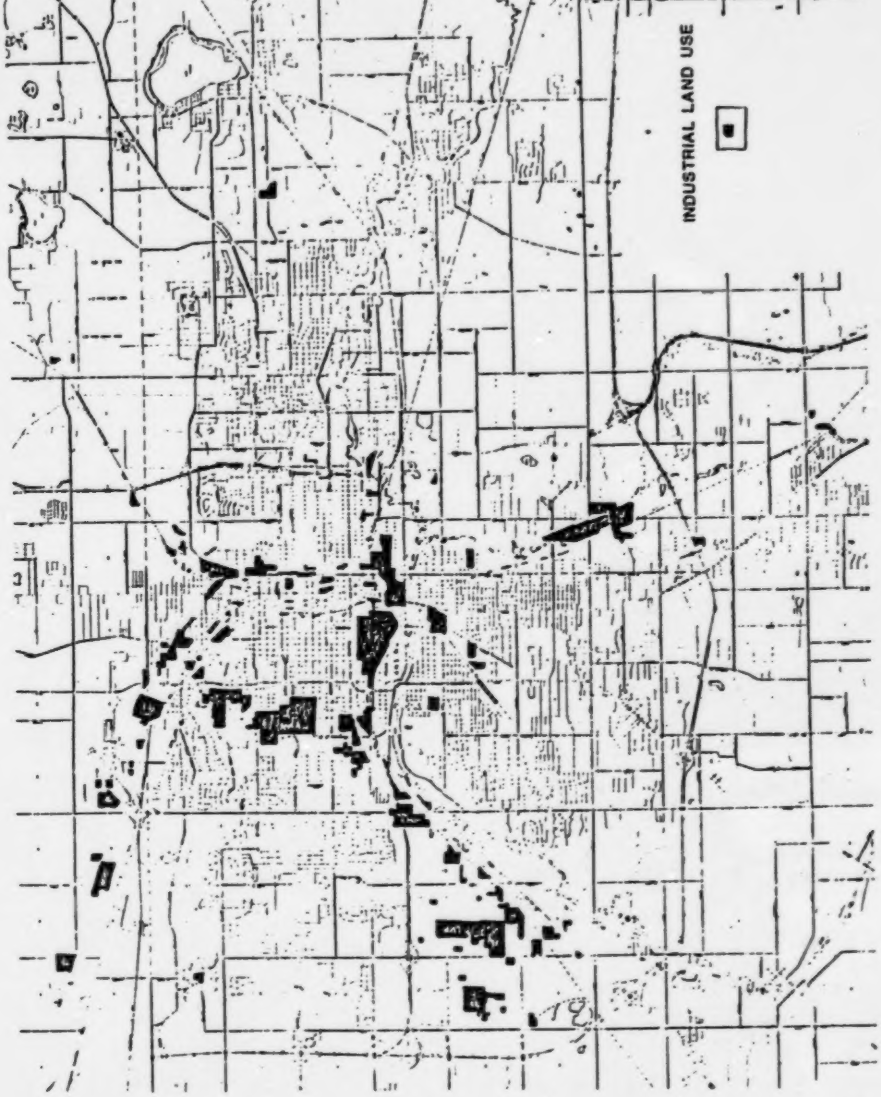
INDUSTRIAL LAND USE
■

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Figure III - 13 Land Use - Industrial

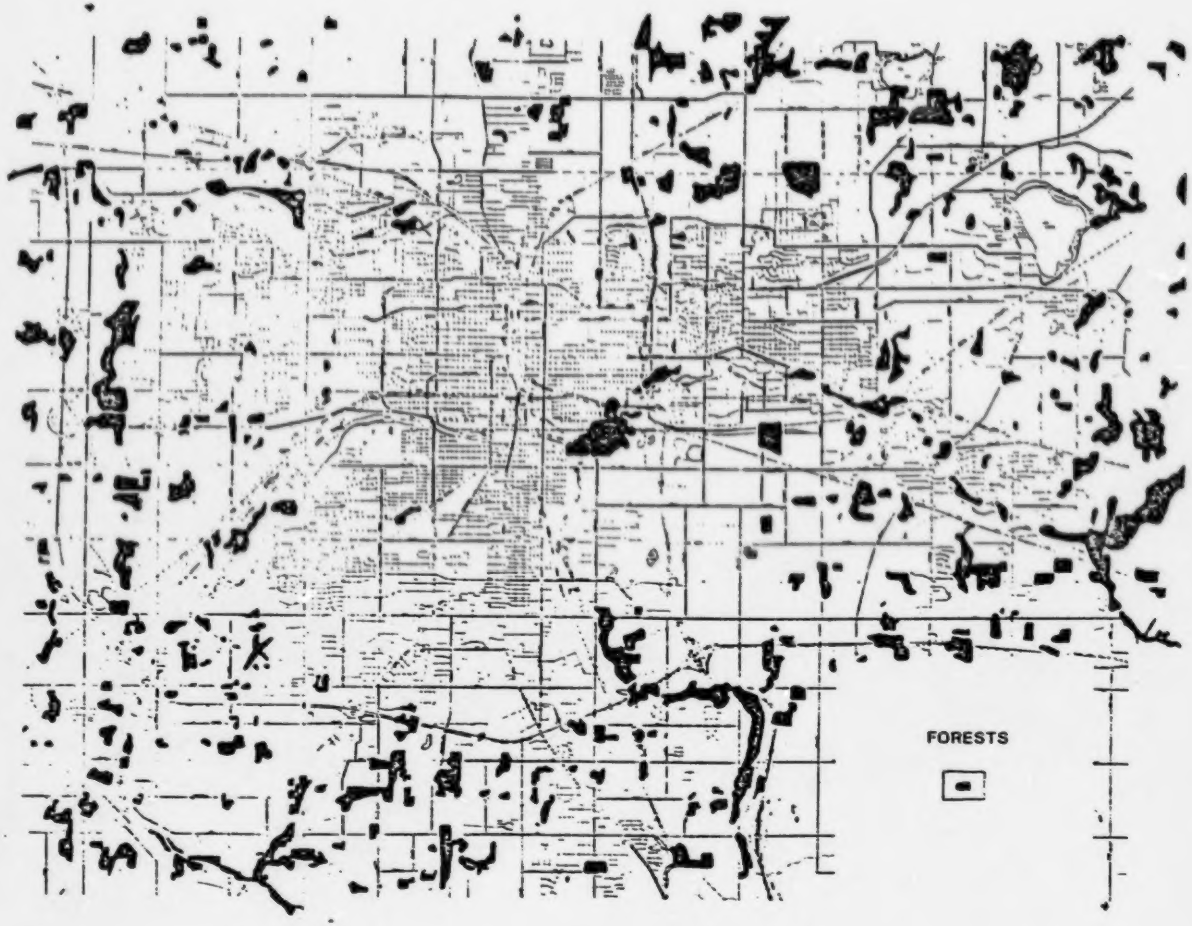

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Figure III - 14 Forested Areas

TABLE III-4

Land Use Designations and Averages For the Lansing Area - Level II

	Category	Acres	Percent
1000	Urban (Total)		
1100	Residential	18,027	55
1200	Commercial	4,686	14
1300	Industrial	1,669	5
1400	Park and Open Space*	8,522	26
1500	Under Construction	-0-	-0-
	TOTAL	32,904	100

* Includes transportation and extractive.

BOGUS SWAMP DRAIN DRAINAGE DISTRICT - LEVELS III AND IV

General

The study area is located in the Bogus Swamp Drain Drainage District, which is located in the Charter Township of Lansing, Ingham County, Michigan (Figure III-15). The Drainage District is west of, and contiguous to, the city of Lansing, and representative of developed urban conditions in the City of Lansing and the larger urban region. The Drainage District is tributary to the Grand River.

The terrain is typical Michigan glacial landscape with gently rolling topography and relatively low slope; the surface elevation at the headwaters is 890 feet and at the outlet 870 feet. The district is 450 acres and much alteration of the land surface has occurred as the district has developed with urban uses. A major part of the topographic changes were undertaken to facilitate residential, commercial and industrial development, on a subdivision or major-site basis.

Drainage System

Storm water runoff in the district is conveyed in a totally enclosed tile system of various sizes (See Figure III-16). All of the storm drains are designed to be physically separate form the sanitary lines. Figure III-17

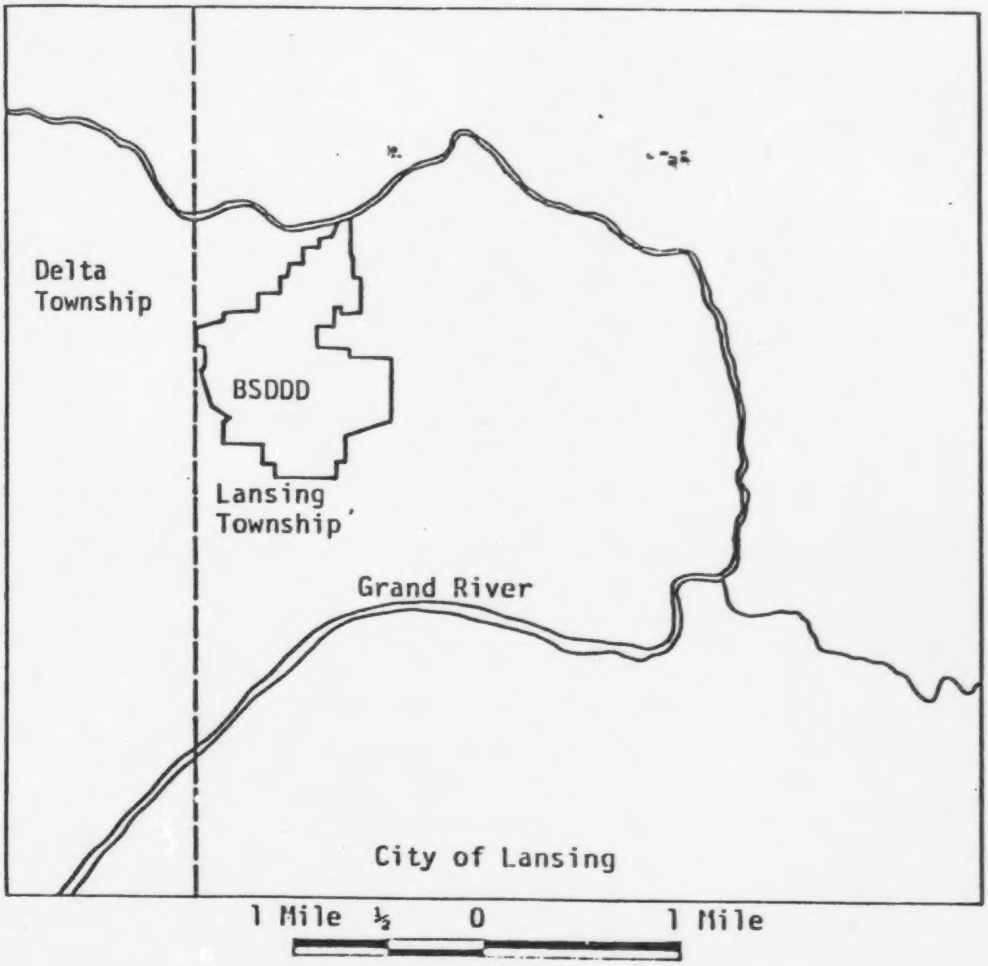
Delta Township
BSDDD
Lansing Township
Grand River
City of Lansing
1 Mile
½
0
1 Mile
N

FIGURE III-15 Location of the Bogus Swamp Drain Drainage District (BSDDD)

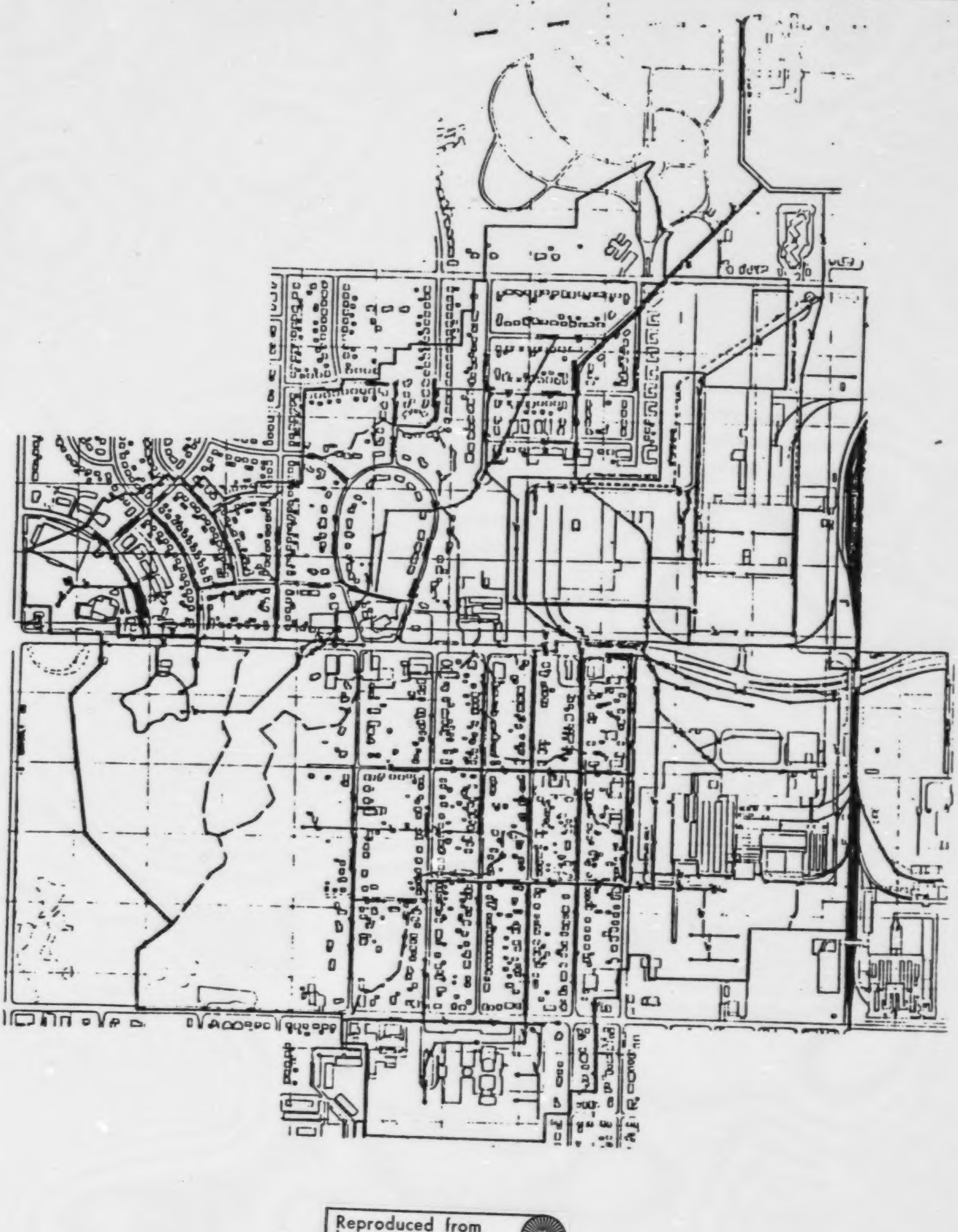

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FIGURE III-16 Storm Drainage System

1SS - Swales and
Ditches

1SC - Curb and
Gutter

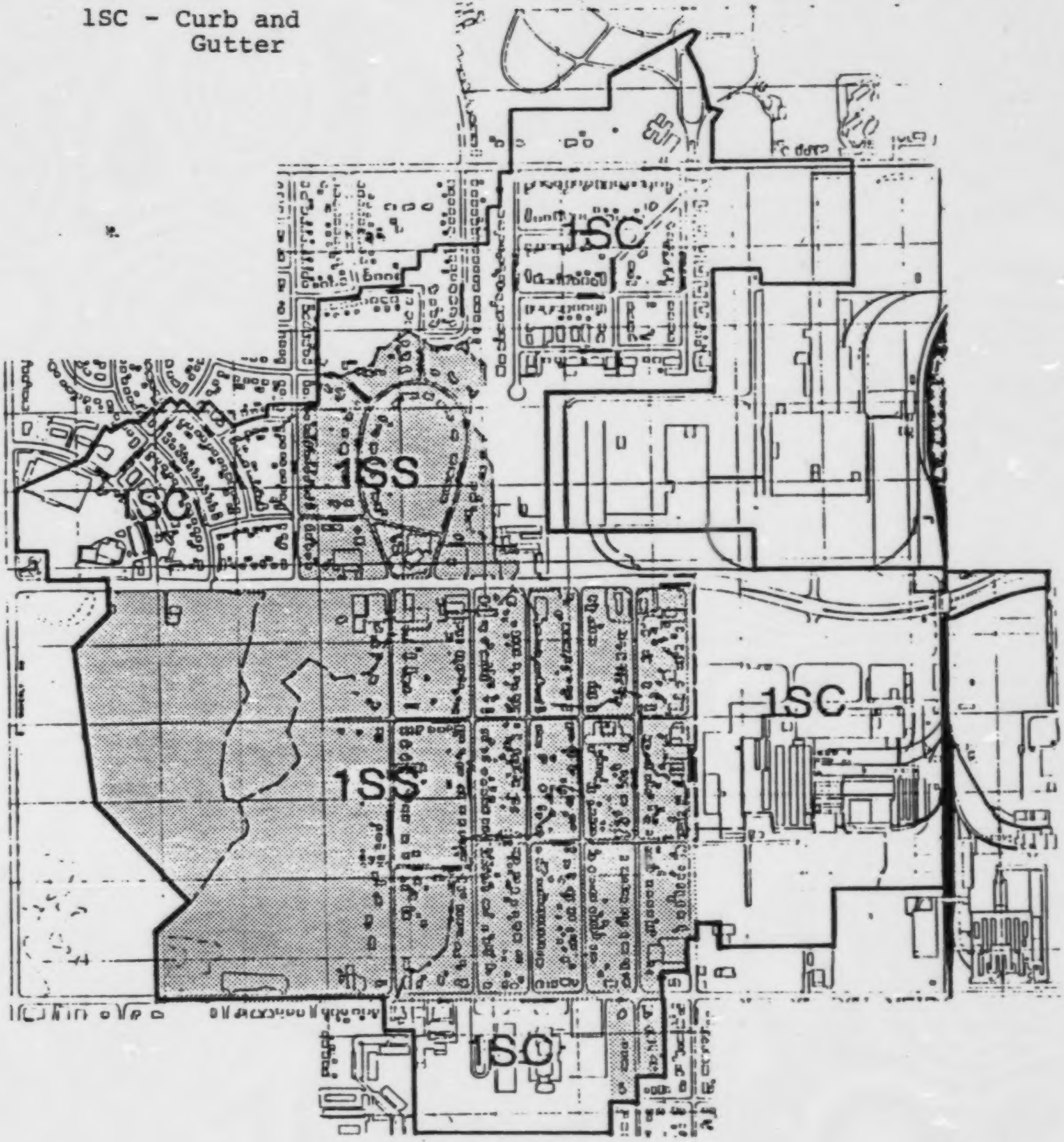

1SC
1SS
SC
1SC
1SS
1SC

FIGURE III-17 Drainage System

presents the various types of drainage systems in the area using NURP designated categories. As can be seen, the study area is made up of the following two categories: Urban-Storm Sewers-Curb and Gutter and Urban-Storm Sewers-Swales and Ditches.

Significant modification of the District's land surface has occurred, resulting in increased impervious surface and decreased infiltration. Major secondary and residential streets, rooftops of heavy-industrial facilities, and commercial strip developments have reshaped the storm hydrograph to one with a steeper rising climb and higher peak. One area, a golf course, has been retained as open space and allows infiltration to occur. Other pervious surfaces are concentrated in those sub-districts having residential subdivisions with front, side and back lawns. One older subdivision (greater than 45 years old) has somewhat more space around the dwellings than the new single-family subdivision which is typical of Lansing and other cities in the Midwest. A multifamily, high density residential development exists in the headwater area and has retained much open space and a wetland. Depression storage is very low in much of the drainage sub-districts within the Bogus Swamp Drainage District. Subdivisions with grassed roadside ditches, and one area where lots are at or slightly below the elevation of the road, offer the most depression storage.

Soils

The Soil Conservation Service has surveyed the soil types in the Lansing area.(4) Data from this survey was utilized to develop a soil profile for the urban area. The soil types in the study area, using the four major hydrological soil groups as defined by the NURP guidelines. all fall into the Hydrologic Group B category (Table III-5). Therefore, to provide more detail the actual soil types for the study area are presented in Figure III-18.

Land Use - Bogus Swamp Drain Drainage District

Land uses and land covers in the Bogus Swamp Drain Drainage District tend to be separated into more or less homogeneous covers which correspond to drainage subdistricts. Examples of most urban uses occur in the drainage district. These land uses include single family residential, multi-family residential, shopping center-commercial, heavy industry and manufacturing. institutional, parkland and open space.

An extensive data gathering effort to define the physical conditions in each of these subdrainage districts was undertaken in accordance with the NURP program guidelines. A set of fixed site data was developed for each drainage area and this information has been computerized by the NURP project team. The fixed site data base includes the following types of information:

Catchment area
Catchment population

TABLE III-5

NURP HYDROLOGIC SOIL GROUPS

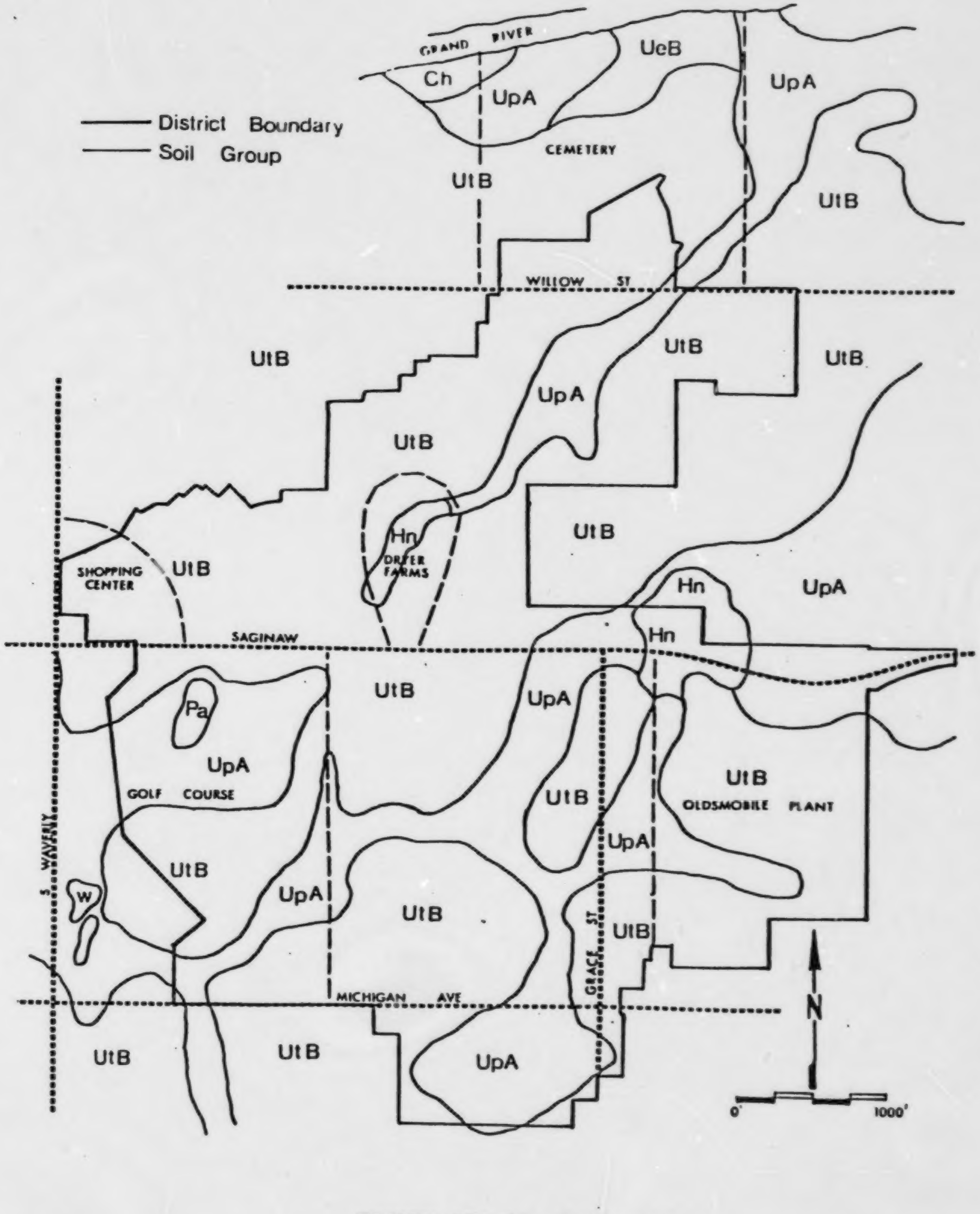
Hydrologic Group A -(low runoff potential). Soils that have high infiltration rates even when thoroughly wetted and a high rate of water transmission.

Hydrologic Group B -(moderately low runoff potential). Soils that have moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission.

Hydrologic Group C -(moderately high runoff potential). Soils that have slow infiltration rates when thoroughly wetted and a slow rate of water transmission.

Hydrologic Group D -(high runoff potential). Soils that have slow infiltration rates when thoroughly wetted and a slow rate of water transmission.

Data Management Procedures Manual, Nationwide Urban Runoff Program, Environmental Protection Agency, March, 1980.



District Boundary
Soil Group
GRAND RIVER
Ch
UpA
UcB
UpA
CEMETERY
UtB
UtB
WILLOW ST
UtB
UtB
UtB
UpA
UtB
UtB
Hn
DRYER FARMS
SHOPPING CENTER
UtB
Hn
UpA
Hn
SAGINAW
Pa
UtB
UpA
UpA
GOLF COURSE
UtB
UtB
OLDSMOBILE PLANT
UpA
S. WAVERLY
W
UtB
UpA
UtB
GRACE ST
UtB
MICHIGAN AVE
N
UtB
UtB
UpA
0'
1000'

FIGURE III-18 Soils Map

SOILS KEY

Map Symbol		Soil Name
Ch		Cohoctah
Hn		Houghton
Pa		Palms
UeB	Urban Land	Bayer, Spinks
UpA	Urban Land	Capac, Colwood
UtB	Urban Land	Marlette

Soil Survey of Ingham County, Michigan, United States Department of Agriculture Soil Conservation Service, August, 1979.

Catchment slope
Drain channel slope
Length of channel

Separate sewer area with curb and gutter or
swales/ditches

Lane miles
Road types and conditions
Landfill areas
Eroding stream banks lengths

Land use (Level III detail)
Impervious area by land use

Hydrological soil group
Pervious area by soil group
Water capacity
Permeability
Erodibility
pH

A full listing of this information by catchment is presented in the separate data report prepared for this project.

The land use data presented in this section of the report is formatted according to the four level reporting sequence prescribed in the Data Management Procedures Manual. Levels III and IV represent District and Sub-district data. Because the subdistrict study area monitored at the main outfall (Station 1) encompasses the entire district. data provided for this area is at Level III. Data for all other sub-districts study areas is at Level IV detail. The techniques used to derive the fixed site data is presented in Table III-6.

Figure III-19 presents the land uses for the study area at Level III detail. Land use codes for all levels. as designated by NURP, are provided in Table III-7. In addition, the Best Management Practices (BMP's). storm water monitoring stations, atmospheric sampling stations and rain gauge locations are presented on this map.

Major sub-district monitoring was provided by the four BMP outfalls and two non-BMP outfalls. Each of the four BMP's were monitored by means of two sampler installations (influent and effluent) with one installation at each of the non-BMP outfalls (total of ten stations).

Level IV data is provided on a station by station basis and covers features of the upstream and adjacent contributing areas for each sampling station. It should be noted that the influent and effluent monitoring stations at each in-line upsized pipe BMP drain equivalent areas. Therefore, land use data provided for BMP inlet stations (10 and 8), applies to BMP outlet stations (9 and 7) as well. However, because of individual area drainage contributions at the in-line wet retention basin (Stations 5 and 6). and the off-line dry detention basin (Stations 3 and 4), the land use data is provided separately for the inlet and outlet station at these BMP's. Non-BMP stations, (Stations 1 and 2), are described separately as well.

Table III-6

Techniques Used to Derive Fixed Site Data

Variable	Technique
Catchment Area	- planimetered - map scale: 1" = 300'
Impervious Area	- planimetered - map scale: 1" = 100'
Effective Impervious Area	- ground proof - areas grouped into 0, 25. 50. 75 and 100% categories
Lane-Miles	- planimetered - map scale: 1" = 100'
Population	- 1980 census (2.27 persons/residence)
Basin Slope	- map scale 1" = 100': 2' contour interval - $S = (CIL/A)$ where: T = length of contour line, A = area, C = Conversion Factor.
Length of Main-Channel	- planimetered - map scale: 1" = 300'
Slope of Channel	$\frac{l_1 \text{ x } elev_1 + \ldots + l_n\ elev_n + l_n + 1 \text{ x } elev_n \text{ x } eleve_{n+1}}{l, \ldots + ln + ln + 1}$ - where l = length of channel
Road Type and Condition	- ground proof - planimetered (map scale: 1" = 300')
Soil Type	- Soil Survey of Ingham Co. Michigan. USDDA, SCS.
Land Use	- planimetered (map scale: 1" = 300 - ground proof

60

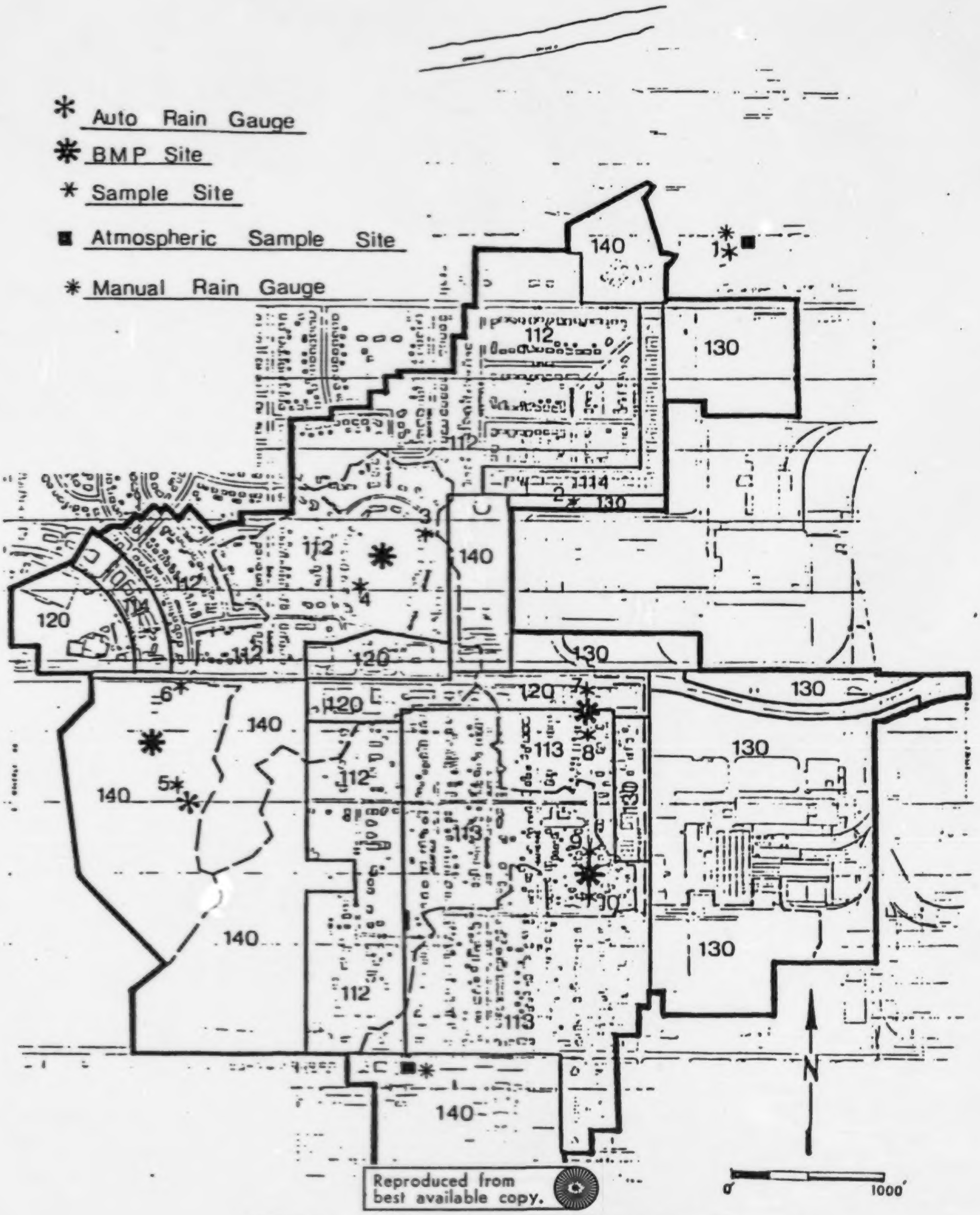

Auto Rain Gauge
BMP Site
Sample Site
Atmospheric Sample Site
Manual Rain Gauge
140
1
130
112
112
114
2
130
3
140
112
4
112
114
120
112
120
130
130
6
120
7
120
140
8
113
130
140
5
112
130
113
130
10
140
130
112
113
140
N
0'
1000'

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FIGURE III-19 Level III Land Use Map

Table III-7

LAND USE DESIGNATIONS

LEVEL I	LEVEL II	LEVEL III	LEVEL IV
1. Urban	11. Residential	111. Rural and Pasture (F 1/2)*	(further break-down as required)
		112. Low Density Residential (1/2 - 2)	
		113. Medium Density Residential (3-8)	
		114. High Density Residential (9+)	
	12. Commercial	120. Commercial	1201. Central Business District
			1202. Strip (linear)
			1203. Shopping Center
	13. Industrial	130. Industrial	1301. Light
			1302. Medium
			1303. Heavy
	14. Parkland	140. Parkland	1401. Institutional
			1402. Parks
	15. Open Space	150. Idle or Vacant	
	16. Other	160. Under Construction	
2. Agriculture			
3. Rangeland			
4. Forest			
5. Water	51. Streams/Canals		
	52. Lakes		
	53. Reservoirs		
	54. Bays/Estuaries		
	55. Oceans		
6. Wetlands			
7. Barren			
8. Other			

*Numbers in parentheses = number of dwelling units/acre.

The following sections provide a brief description of each sub-district and presents a summary of the land use data and catchment characteristics. Data is provided for upstream stations first, followed by their corresponding downstream stations as shown in Figure III-20.

Sub-district Study Area Monitored at Stations 9 and 10

The location of this sub-district in relationship to the entire drainage district is presented in Figure III-21. The sub-district is 74.9 acres in size with 28.1 acres of it being impervious. (Effective impervious areas make up 23.2 acres.) It is comprised of a mixture of low and medium residential, heavy industrial and institutional land uses as shown in Figure III-22 and Table III- 8 . The runoff from this sub-district was monitored during rainfall events at Stations 10 and 9, the influent and effluent stations of the first upsized pipe BMP located on Grace Street.

Sub-district Study Area Monitored at Stations 7 and 8

The area that draws into stations 7 and 8 (upsized pipe BMP) is shown in Figure III-23. The area of this sub-district is 163.9 acres. It should be remembered that this area is cumulative and includes the 74.9 acres from Station 9 and 10 sub-district. Of the 163.9 acres in this sub-district, 45.1 acres are impervious and only 34.4 acres are effective impervious. The district is comprised of low and medium residential, commercial, heavy industrial and institutional land uses as shown in Figure III-24 and Table III- 9.

FIGURE III-20 STATION FLOW DIAGRAM

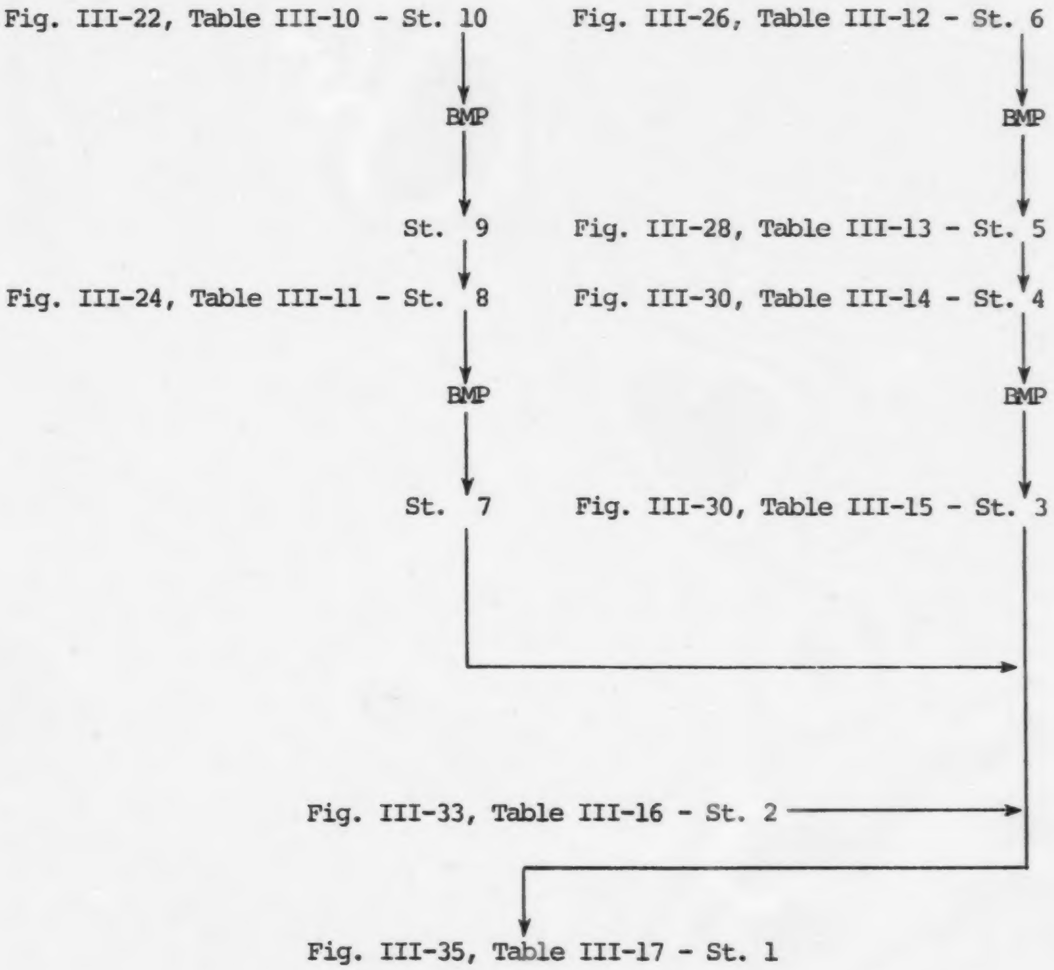
Fig. III-22, Table III-10 - St. 10
BMP
St. 9
Fig. III-24, Table III-11 - St. 8
BMP
St. 7
Fig. III-26, Table III-12 - St. 6
BMP
Fig. III-28, Table III-13 - St. 5
Fig. III-30, Table III-14 - St. 4
BMP
Fig. III-30, Table III-15 - St. 3
Fig. III-33, Table III-16 - St. 2
Fig. III-35, Table III-17 - St. 1

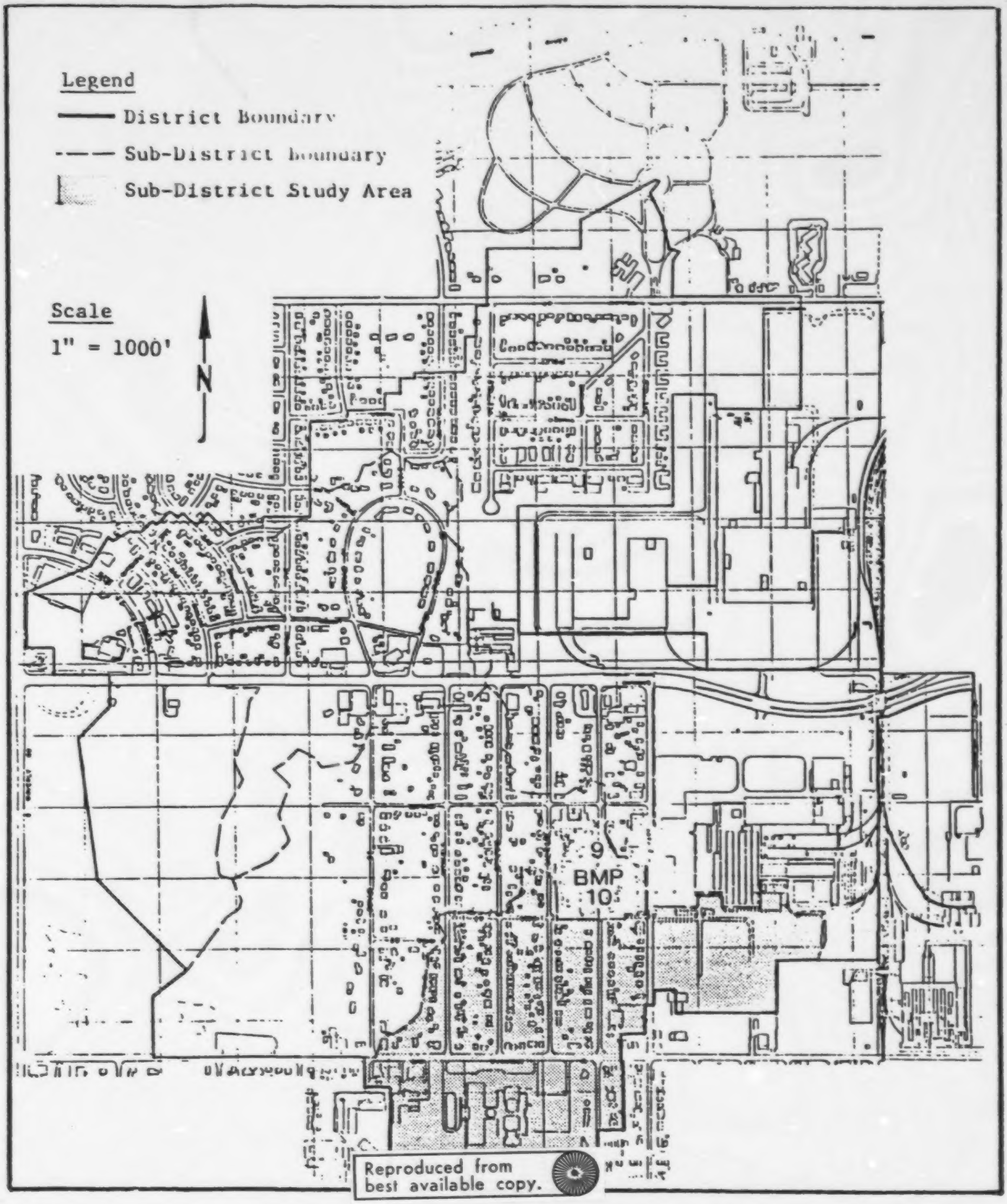
Legend
District Boundary
Sub-District Boundary
Sub-District Study Area
Scale
1" = 1000'
N
9
BMP
10
Reproduced from best available copy.

FIGURE III-21 SUB-DISTRICT STUDY AREA MONITORED AT STATIONS 9 AND 10 (In-line Upsized Pipe BMP)

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best available copy.

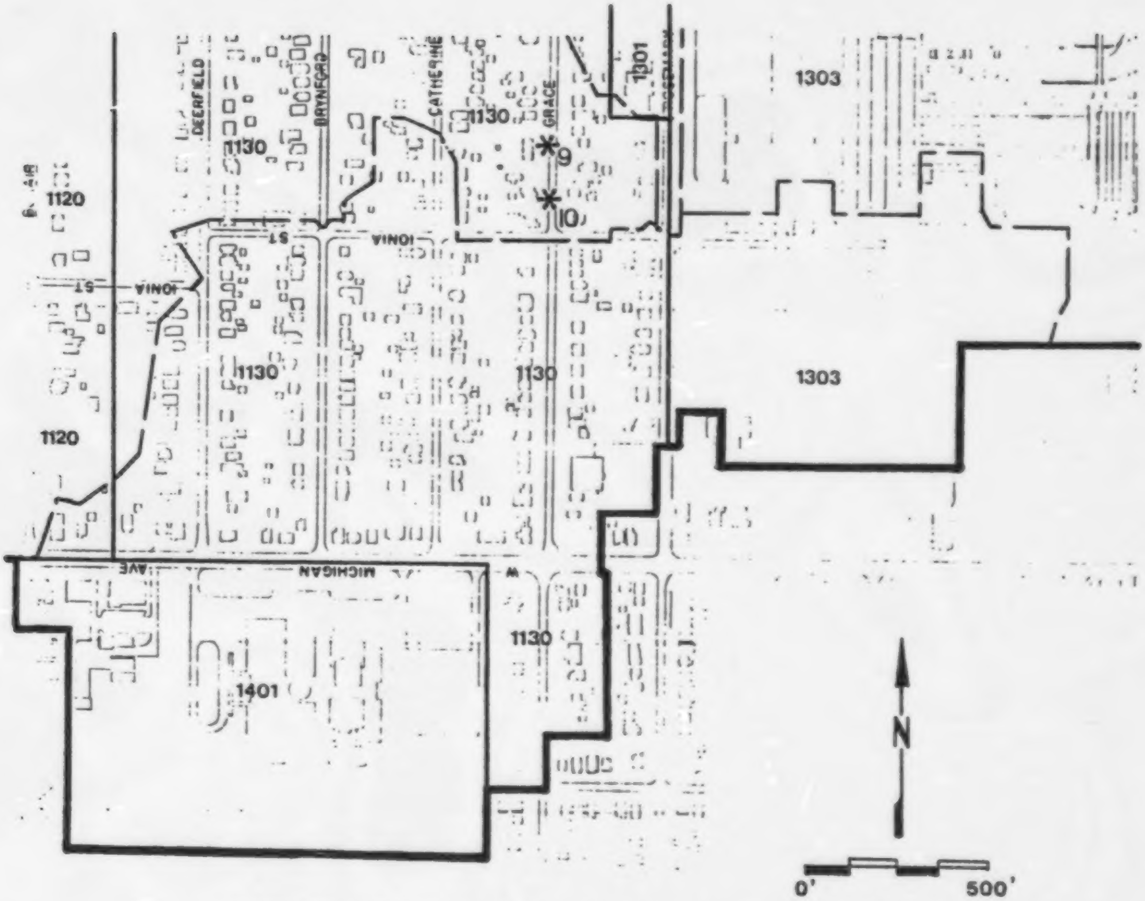

DEERFIELD
BRYNFORD
CATHERINE
GRACE
ROSEMARY
1301
1303
1130
9
10
1120
IONIA ST
MICHIGAN AVE
1303
1130
1120
1130
1401
1130
N
0'
500'

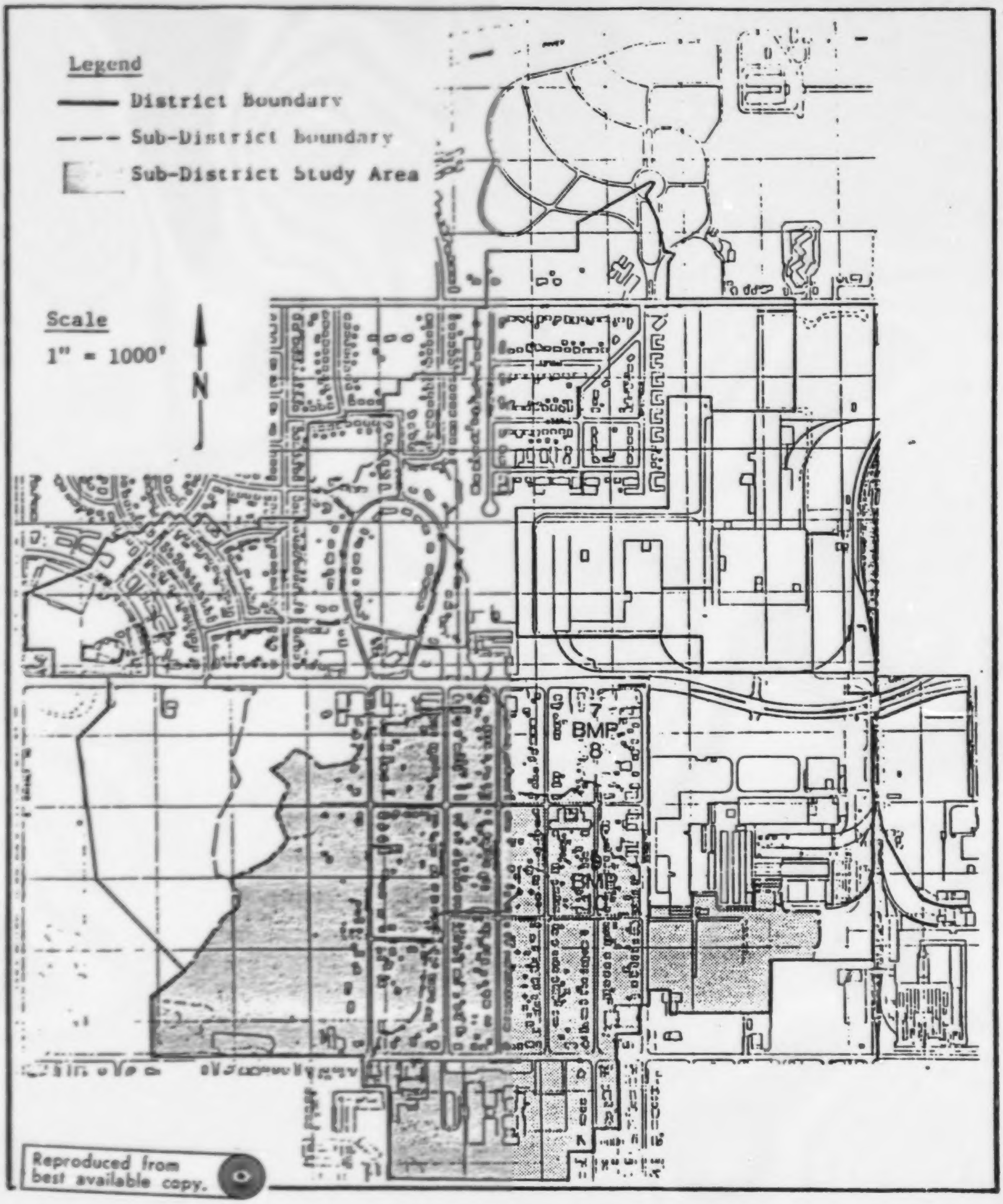
FIGURE III-22 Level IV Land Use Map
Sub-district Monitored at
Stations 9 and 10

TABLE III- 8

Features of the Sub-District Study Area Monitored at Stations 9 and 10 (In-line Upsized Pipe BMP)

		Cumulative Area (acres)		
	Land Use	Area	Impervious Area	Effective Impervious Area
1120	Residential (.5 - 2 units/acre)	1.8	1.3	.70
1130	Residential (2.5 - 8 units/acre)	33.9	10.3	6.0
1140	Residential (8 units/acre)			
1202	Commercial (Linear strip)			
1203	Commercial (Shopping center)			
1301	Industrial (light)			
1303	Industrial (heavy)	16.4	7.8	7.8
1400	Parkland			
1401	Institutional	22.8	8.7	8.7
	Total	74.9	28.1	23.2

Catchment Population	370
Catchment Slope (ft./mi.)	194
Channel Slope (ft./mi.)	63
Length of Channel (ft.)	9,230


Legend
District Boundary
Sub-District Boundary
Sub-District Study Area
Scale
1" = 1000'
N
7
BMP
8
BMP

Reproduced from best available copy.

FIGURE III-23 SUB-DISTRICT STUDY AREA MONITORED AT STATIONS 7 AND 8
(In-line Upsized Pipe BMP)

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best available copy.

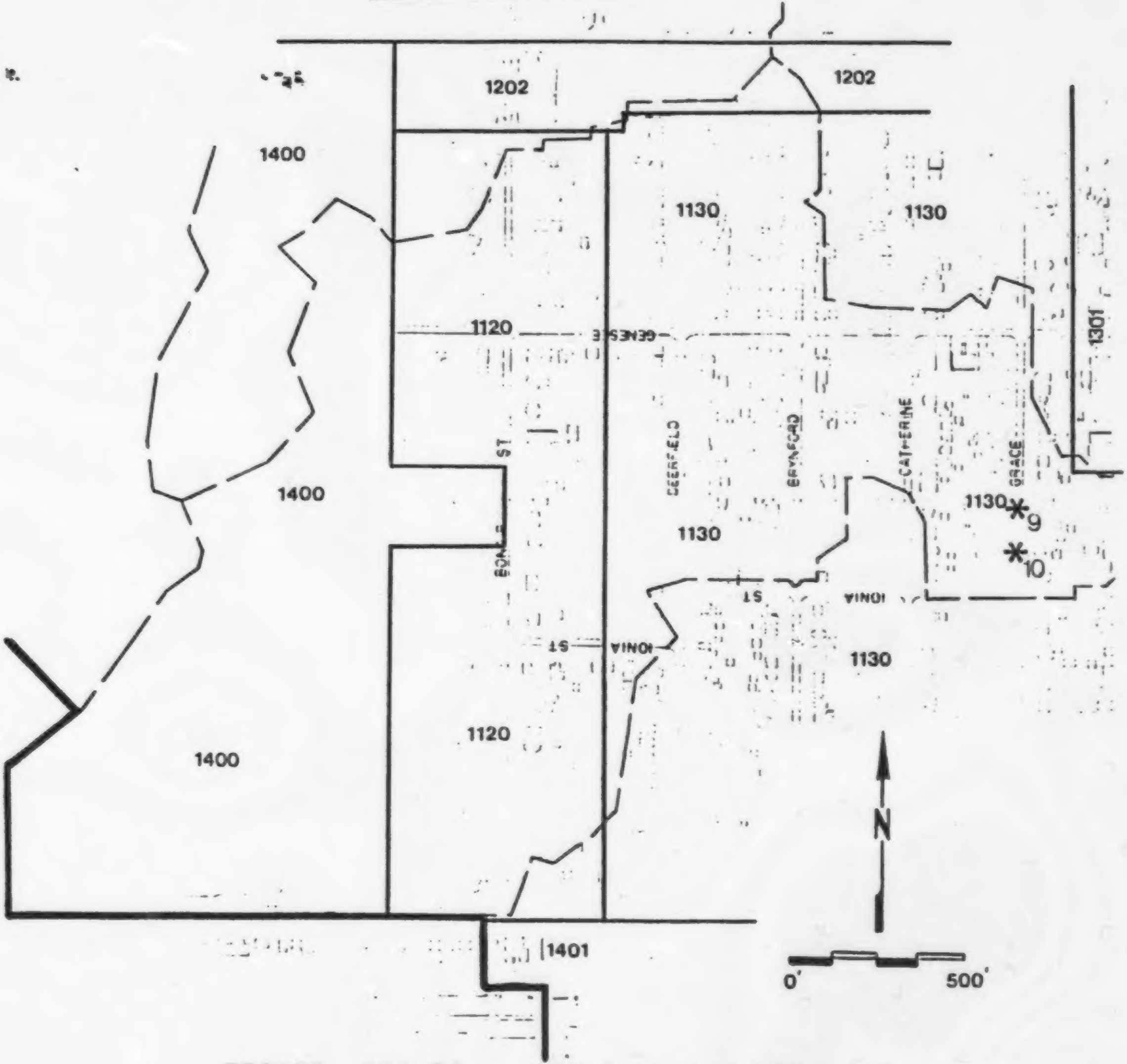
1202
1202
1400
1130
1130
1120
GENESEE
1301
ST
DEERFIELD
BRYNFORD
CATHERINE
GRACE
1400
1130
1130
9
10
ST
IONIA
IONIA
ST
1130
1120
1400
N
0'
500'
1401

FIGURE III-24 Level IV Land Use Map
Sub-district Monitored at
Stations 7 and 8

TABLE III- 9

Features of the Sub-District Study Area Monitored at Stations 7 and 8 (In-line Upsized Pipe BMP)

		Cumulative Area (acres)		
	Land Use	Area	Impervious Area	Effective Impervious Area
1120	Residential (.5 - 2 units/acre)	29.3	6.5	4.1
1130	Residential (2.5 - 8 units/acre)	61.6	20.9	12.6
1140	Residential (8 units/acre)			
1202	Commercial (Linear strip)	0.6	0.6	0.6
1203	Commercial (Shopping center)			
1301	Industrial (light)			
1303	Industrial (heavy)	16.4	7.8	7.8
1400	Parkland	33.2	0.6	0.6
1401	Institutional	22.8	8.7	8.7
	Total	163.9	45.1	34.4

Catchment Population	850
Catchment Slope (ft./mi.)	226
Channel Slope (ft./mi.)	48
Length of Channel (ft.)	15,530

Sub-district Study Area Monitored at Station 6

Figure III-25 shows the location of this sub-district in relationship to the entire study area. The sub-district is comprised of a major shopping center and low and high density housing (See Table III-10 and Figure III-26). The sub-district is 30.3 acres in size with 20.5 acres being impervious. The runoff from this area is monitored at Station 6, the inlet to the in-line retention basin BMP.

Sub-district Study Area Monitored at Station 5

The area that is monitored at Station 5, Retention Basin Outlet, is 67 acres in size with 30.3 acres being comprised of the area that first drains through Station 6 (Figure III-27). The additional land that drains into Station 5 is all parkland (City golf course). The in-line wet retention bsain BMP is located on the golf course between Stations 5 and 6. Land use categories for this area are presented in Table III-11 and Figure III-28.

Sub-district Study Area Monitored at Station 4

As can be seen in Figure III-29, the area draining into Station 4 is comprised of the area from Stations 6 and 5 plus an additional 45.7 acres of mixed land uses. These

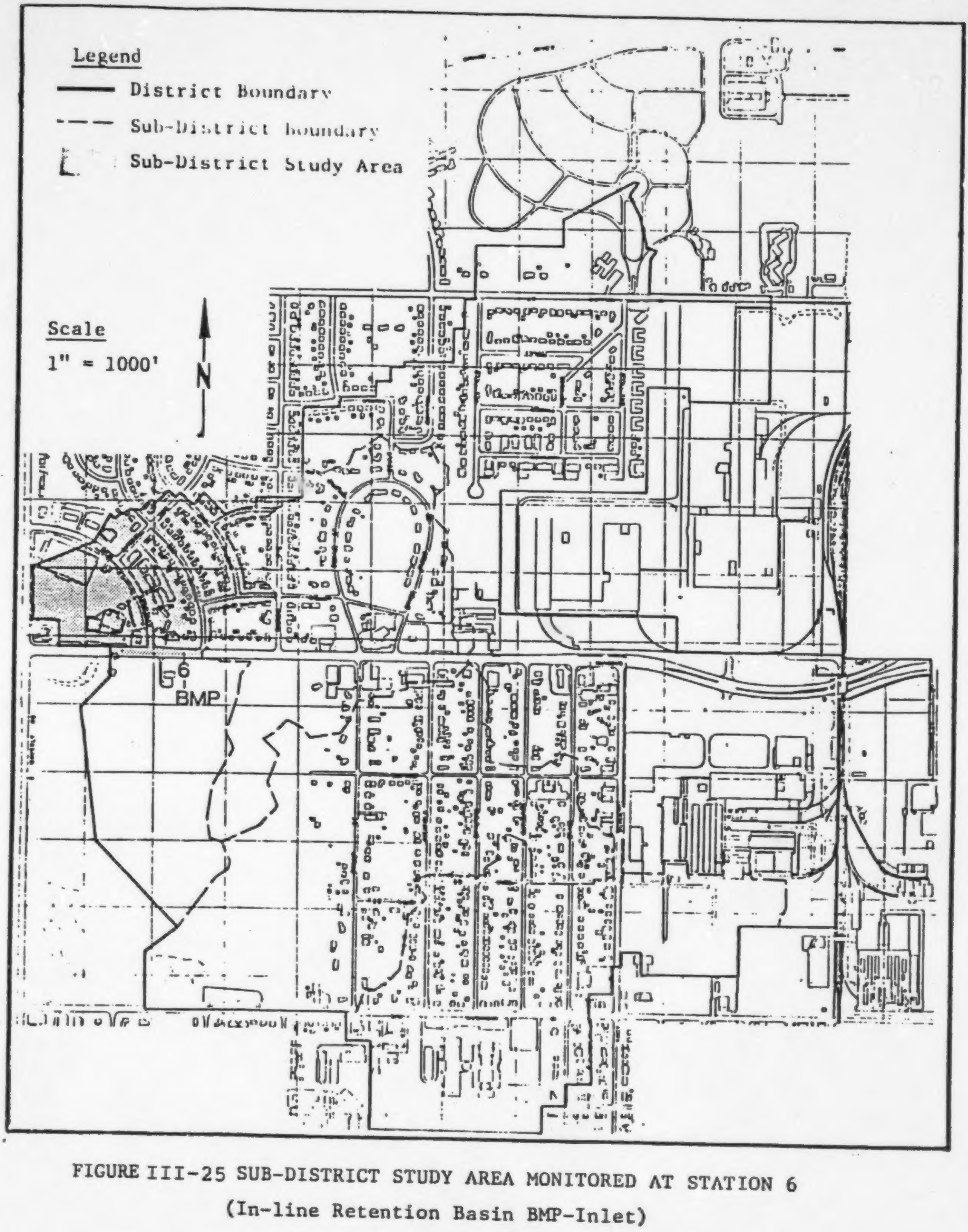
Legend
District Boundary
Sub-District Boundary
Sub-District Study Area
Scale
1" = 1000'
N
6
BMP

FIGURE III-25 SUB-DISTRICT STUDY AREA MONITORED AT STATION 6
(In-line Retention Basin BMP-Inlet)

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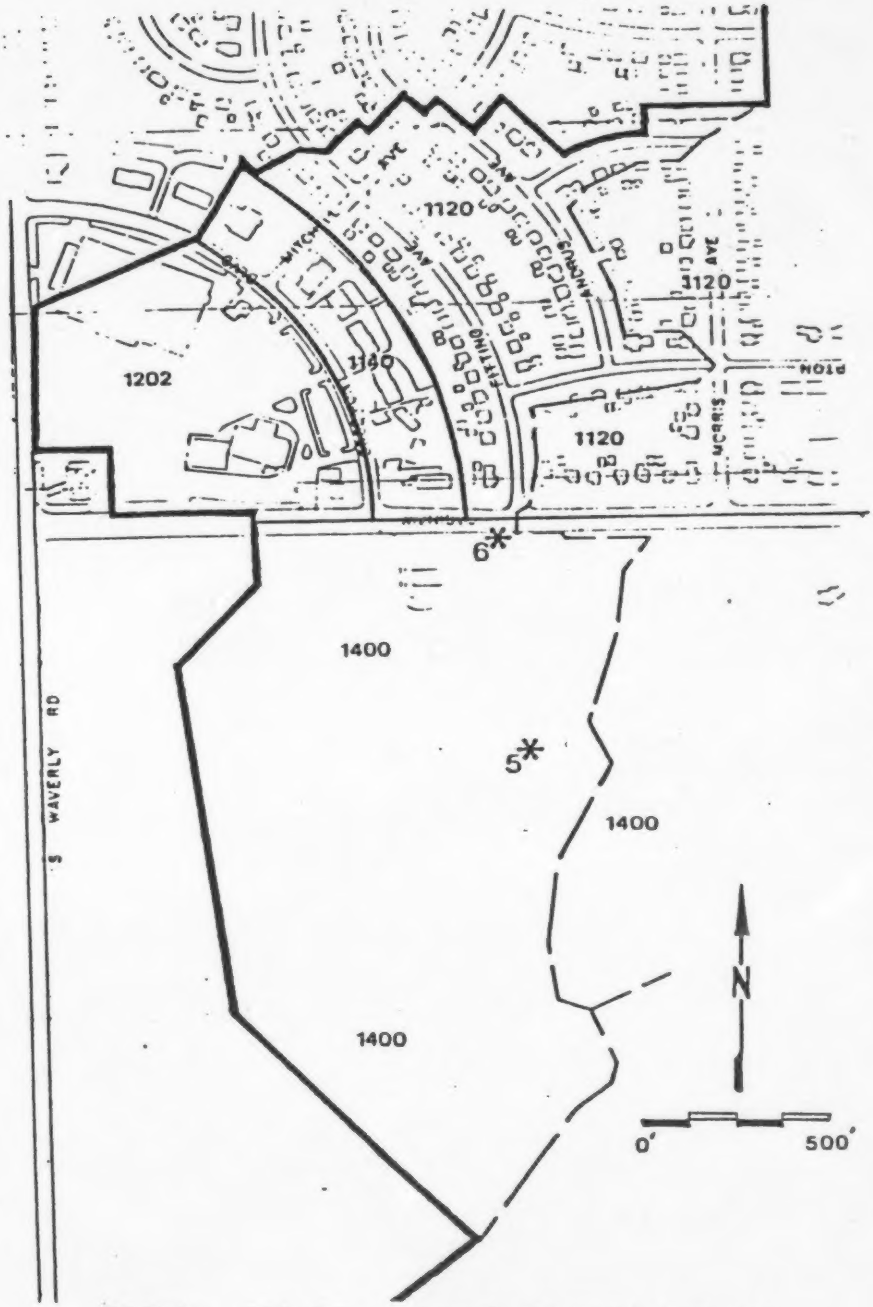
1120
1120
1120
1140
1202
1400
1400
1400
6*
5*
AVE
AVE
AVE
MITCH
FITTING
SIMONS
MCRRIS
S WAVERLY RD
N
0'
500'

FIGURE III-26 Level IV Land Use Map
Sub-district Monitored at
Stations 5 and 6

TABLE III-10

Features of the Sub-District Study Area
Monitored at Station 6
(In-line Retention Basin BMP - Inlet)

	Land Use	Cumulative Area (acres) Area	 Impervious Area	 Effective Impervious Area
1120	Residential (.5 - 2 units/acre)	15.0	7.3	5.0
1130	Residential (2.5 - 8 units/acre)			
1140	Residential (8 units/acre)	5.2	3.1	2.6
1202	Commercial (Linear strip)			
1203	Commercial (Shopping center)	10.1	10.1	10.1
1301	Industrial (light)			
1303	Industrial (heavy)			
1400	Parkland			
1401	Institutional			
	Total	30.3	20.5	17.7

Catchment Population	340
Catchment Slope (ft./mi.)	121
Channel Slope (ft./mi.)	22
Length of Channel (ft.)	4,800

74

Reproduced from best available copy.

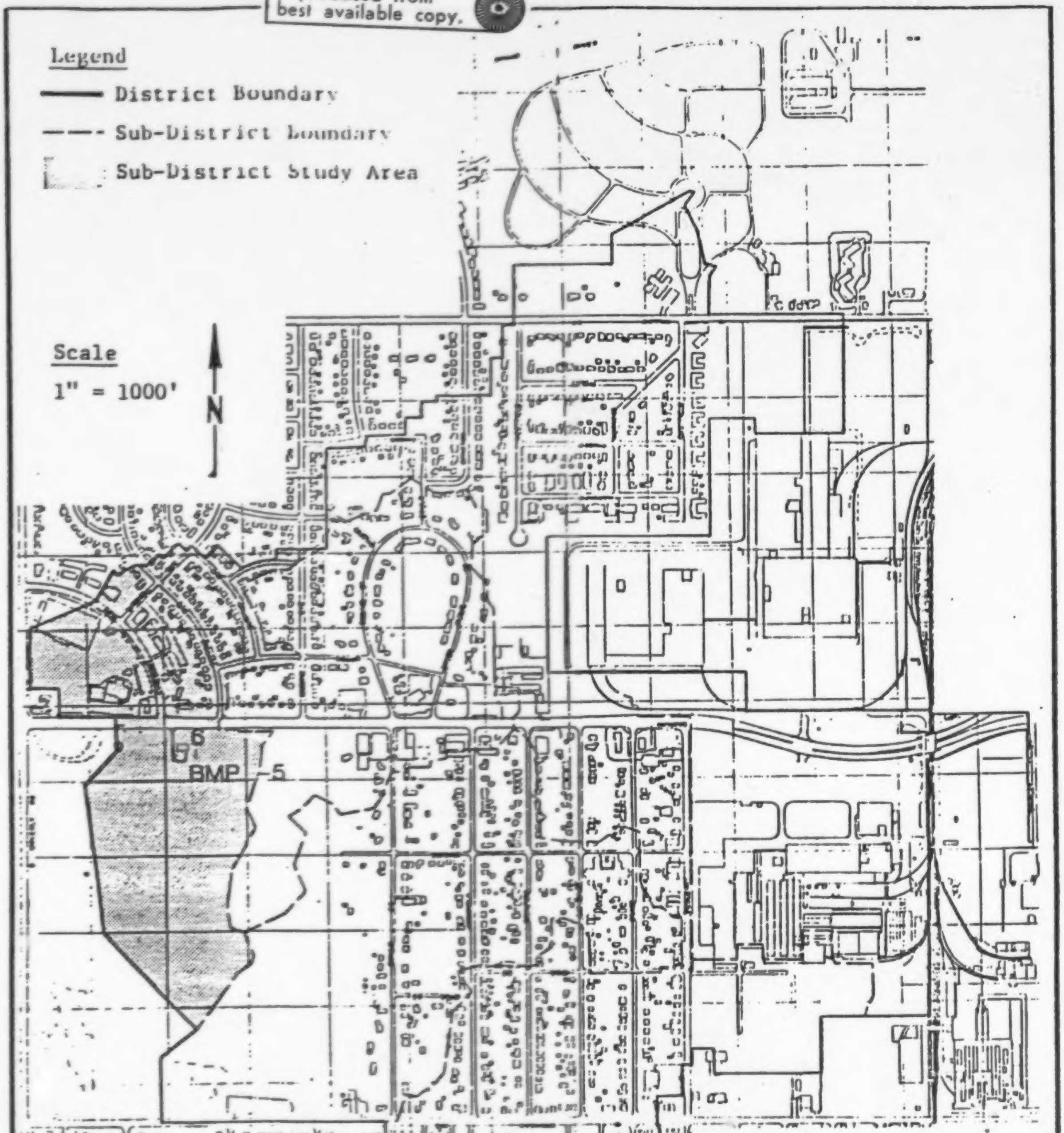
Legend
District Boundary
Sub-District Boundary
Sub-District Study Area
Scale
1" = 1000'
N
6
BMP-5

FIGURE III-27 SUB-DISTRICT STUDY AREA MONITORED AT STATION 5
(In-line Retention Basin BMP-Outlet)

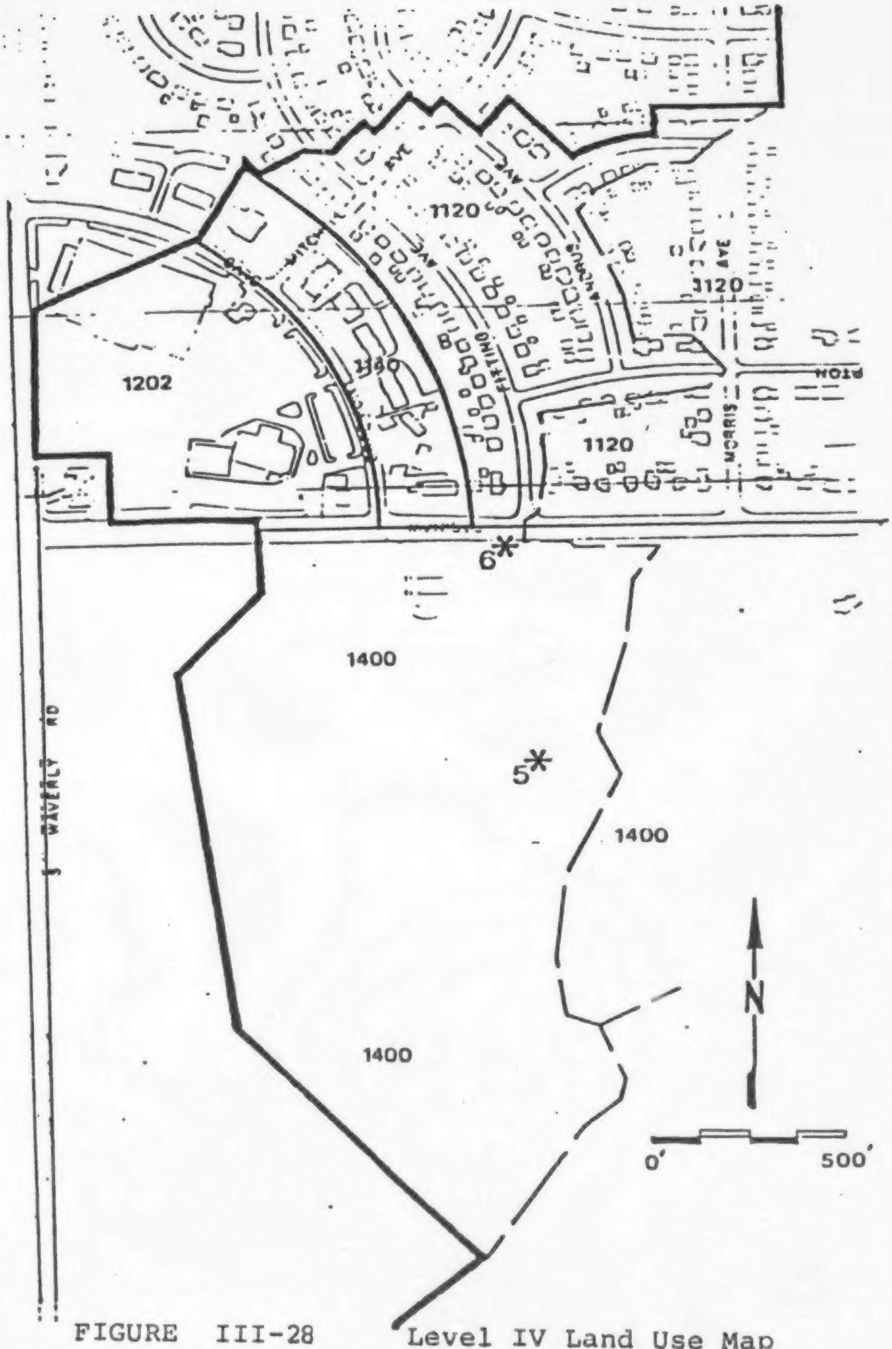
1120
1120
1202
1140
1120
AVE
AVE
AVE
AVE
MORRIS
6*
1400
5*
1400
1400
S WAVERLY RD
N
0'
500'

FIGURE III-28 Level IV Land Use Map Subdistrict Monitored at Stations 5 and 6

TABLE III-11

Features of the Sub-District Study Area Monitored at Station 5 (In-line Retention Basin BMP - Outlet)

		Cumulative Area (acres)		
	Land Use	Area	Impervious Area	Effective Impervious Area
1120	Residential (.5 - 2 units/acre)	15.0	7.3	5.0
1130	Residential (2.5 - 8 units/acre)			
1140	Residential (8 units/acre)	5.2	3.1	2.6
1202	Commercial (Linear strip)			
1203	Commercial (Shopping center)	10.1	10.1	10.1
1301	Industrial (light)			
1303	Industrial (heavy)			
1400	Parkland	36.7	0.6	0.6
1401	Institutional			
	Total	67	21.1	18.3

Catchment Population	340
Catchment Slope (ft./mi.)	200
Channel Slope (ft./mi.)	29
Length of Channel (ft.)	5,480

77

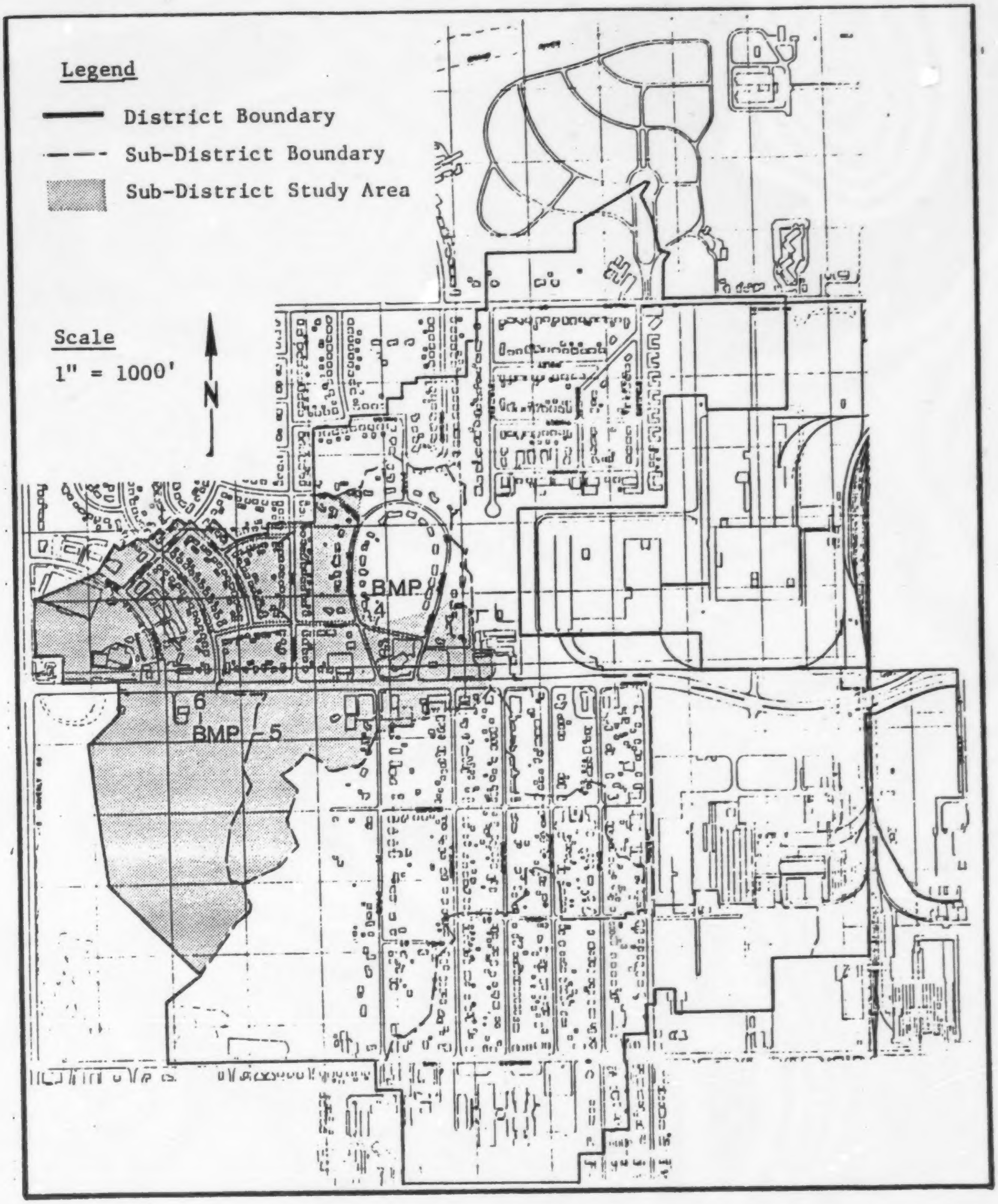
Legend
District Boundary
Sub-District Boundary
▨ Sub-District Study Area
Scale
1" = 1000'
N
BMP
4
6
BMP-5

FIGURE III-29 SUB-DISTRICT STUDY AREA MONITORED AT STATION 4
(Off-line Detention Basin BMP-Inlet)

additional land uses includes 8.4 acres of commercial strip developments, 1.8 acres of institutional uses (school), 12.5 acres of additional parkland and 23 additional acres of low density housing (See Table III-12 and Figure III-30). Station 4 is the inlet station to the off-line detention basin BMP.

Sub-district Study Area Monitored at Station 3

Station 3 is the outlet station to the off-line detention basin BMP. As is the case with Station 4, the area draining into Station 3 is cumulative and includes those areas already described for Stations 6, 5 and 4 (Figure III-31). Only an additional 14.9 acres of low density housing drain into Station 3. Land use descriptions are presented in Table III- 13 and Figure III-32 for this drainage area.

Sub-district Study Area Monitored at Station 2

The 63 acres that drain into Station 2 is comprised solely of the Oldsmobile Assembly Plant. Of the 63 acres. 40.4 acres are impervious. Figure III-33 shows the location of this sub-district in relationship to the entire study area. Table III-14 and Figure III-34 show the land uses for this sub-drainage district.

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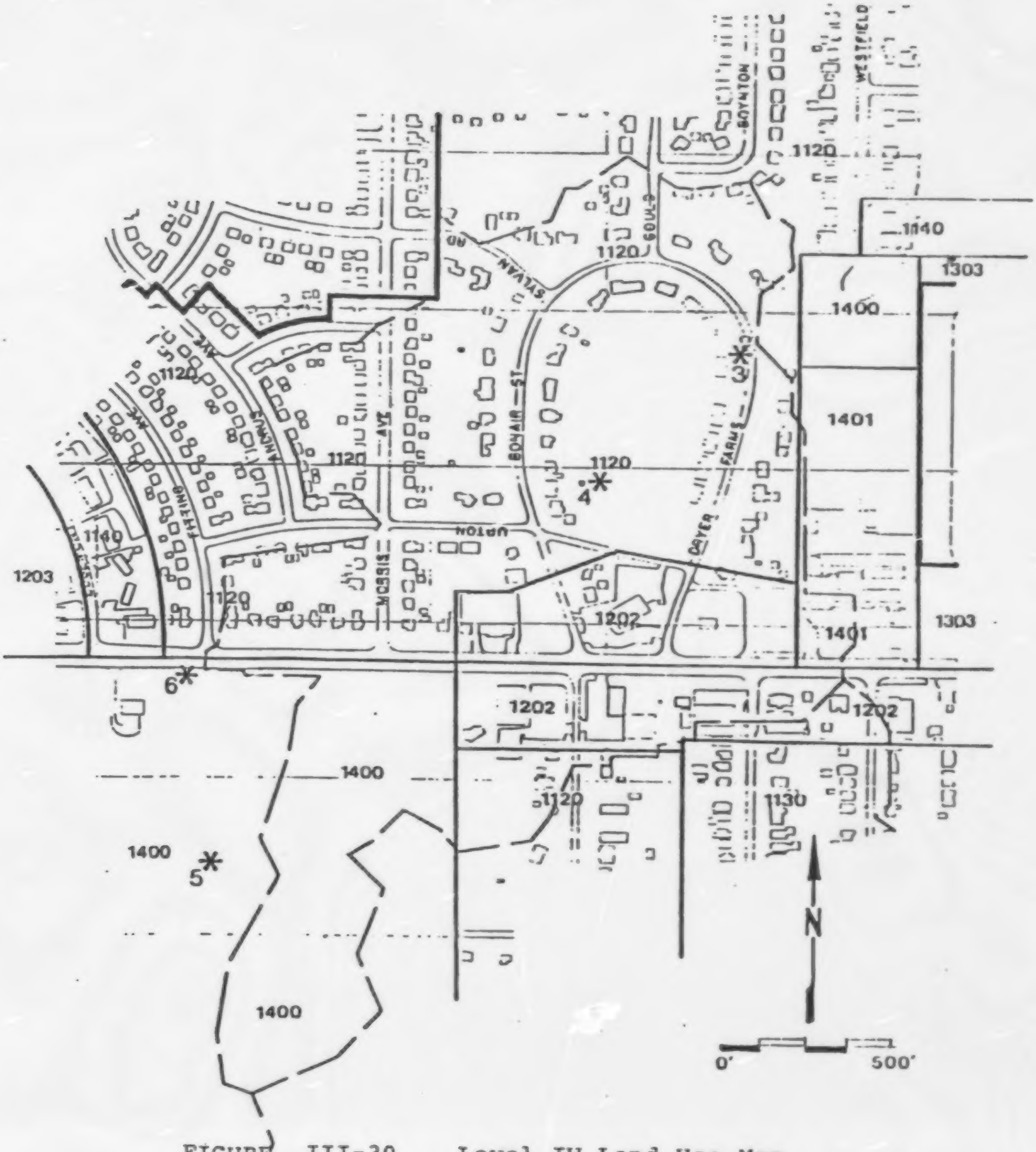
BOYNTON
WESTFIELD
1120
1140
1303
1400
1401
1303
STAVAN
BONAIR ST
FARMS
DYER
URTON
MORRIS
AVE
WHITING
1203
1202
1130
3
4
5
6
N
0'
500'

FIGURE III-30 Level IV Land Use Map
Sub-district Monitored at
Stations 3 and 4

TABLE III-12

Features of the Sub-District Study Area
Monitored at Station 4
(Off-line Detention Basin BMP - Inlet)

		Cumulative Area (acres)		
	Land Use	Area	Impervious Area	Effective Impervious Area
1120	Residential (.5 - 2 units/acre)	38	13.4	8.3
1130	Residential (2.5 - 8 units/acre)			
1140	Residential (8 units/acre)	5.2	3.1	2.6
1202	Commercial (Linear strip)	8.4	4.9	4.9
1203	Commercial (Shopping center)	10.1	10.1	10.1
1301	Industrial (light)			
1303	Industrial (heavy)			
1400	Parkland	49.2	1.2	1.2
1401	Institutional	1.8	1.7	1.7
	Total	112.7	34.4	28.3

Catchment Population	480
Catchment Slope (ft./mi.)	233
Channel Slope (ft./mi.)	32
Length of Channel (ft.)	9,980

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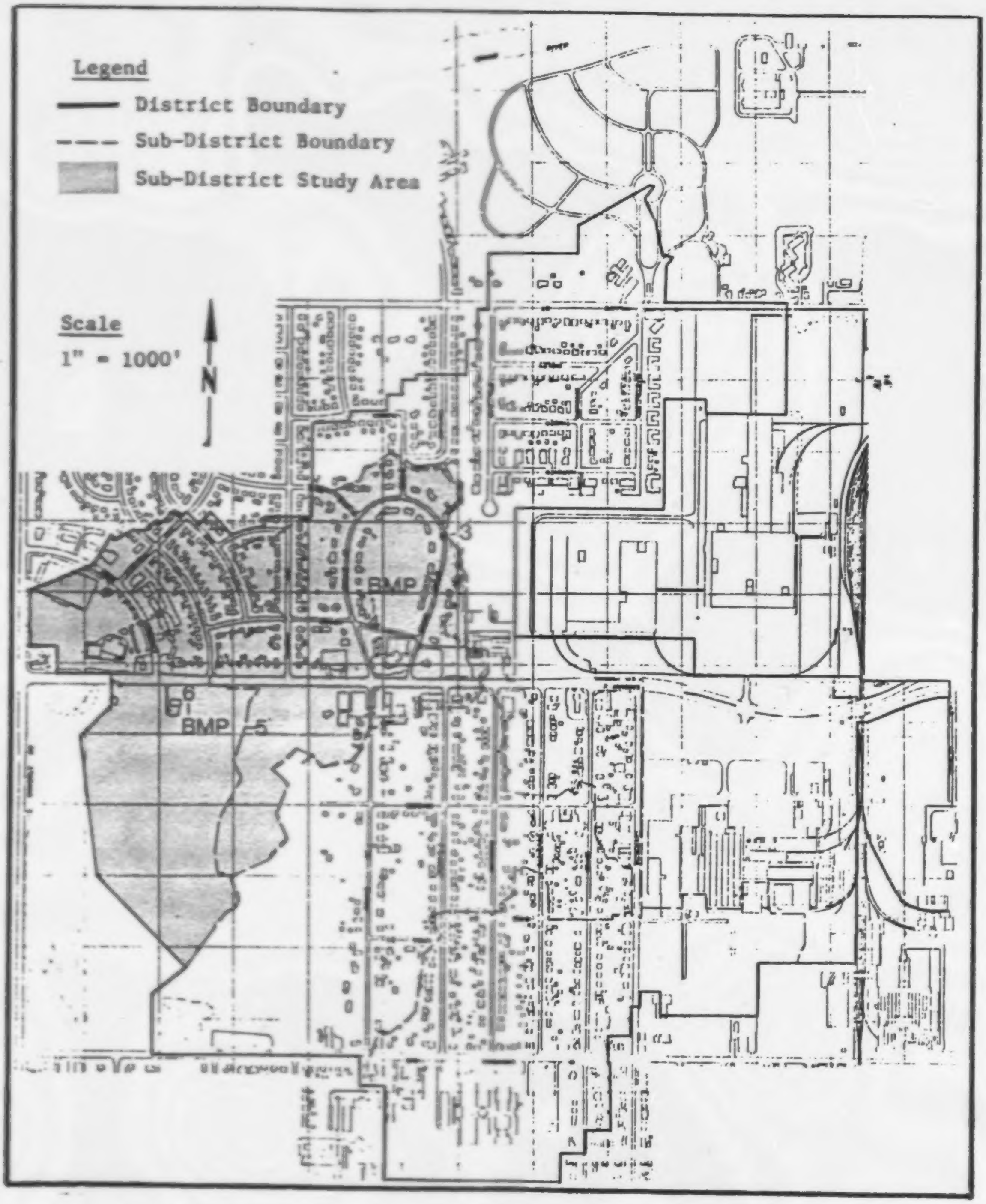
Legend
District Boundary
Sub-District Boundary
Sub-District Study Area
Scale
1" = 1000'
N
BMP
3
6
7
BMP
5

FIGURE III-31 SUB-DISTRICT STUDY AREA MONITORED AT STATION 3
(Off-line Detention Basin BMP-Outlet)

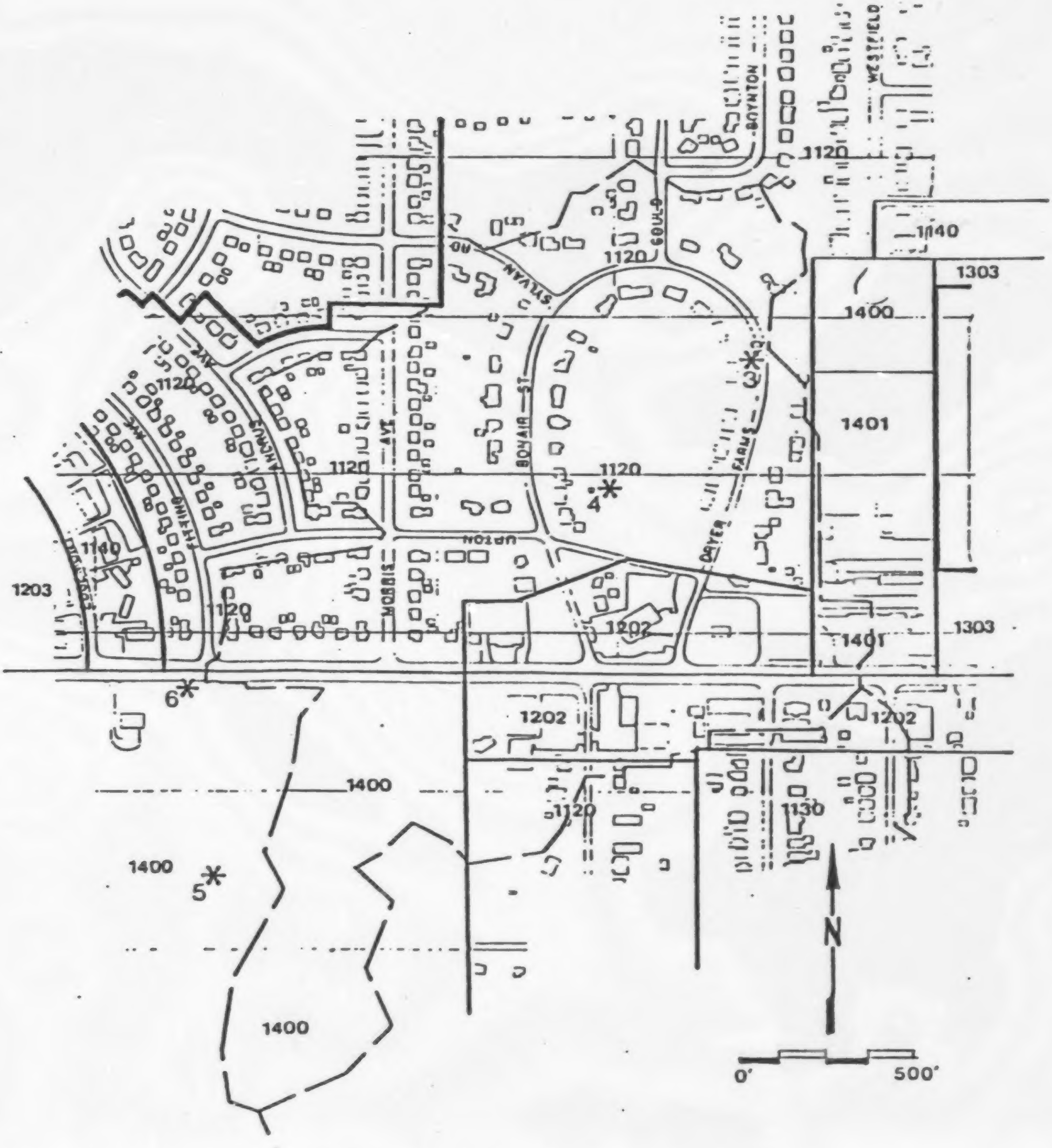
BOYNTON
WESTFIELD
1120
1140
1303
GOULD
SYLVAN RD
1120
1400
1401
BONAIR ST
AVE
MORRIS
1120
1120
1120
4
FARMS
DYER
UPTON
1140
1203
1120
1202
1303
1401
6
1202
1202
1400
1120
1130
1400
5
N
1400
0'
500'

FIGURE III-32 Level IV Land Use Map
Sub-district Monitored at
Station 3 and 4

TABLE III-13

Features of the Sub-District Study Area Monitored at Station 3 (Off-line Detention Basin BMP - Outlet)

		Cumulative Area (acres)		
	Land Use	Area	Impervious Area	Effective Impervious Area
1120	Residential (.5 - 2 units/acre)	52.9	16.1	9.8
1130	Residential (2.5 - 8 units/acre)			
1140	Residential (8 units/acre)	5.2	3.1	2.6
1202	Commercial (Linear strip)	8.4	4.9	4.9
1203	Commercial (Shopping center)	10.1	10.1	10.1
1301	Industrial (light)			
1303	Industrial (heavy)			
1400	Parkland	49.2	1.2	1.2
1401	Institutional	1.8	1.7	1.7
	Total	127.6	37.1	30.3

Catchment Population	550
Catchment Slope (ft./mi.)	121
Channel Slope (ft./mi.)	27
Length of Channel (ft.)	10,650

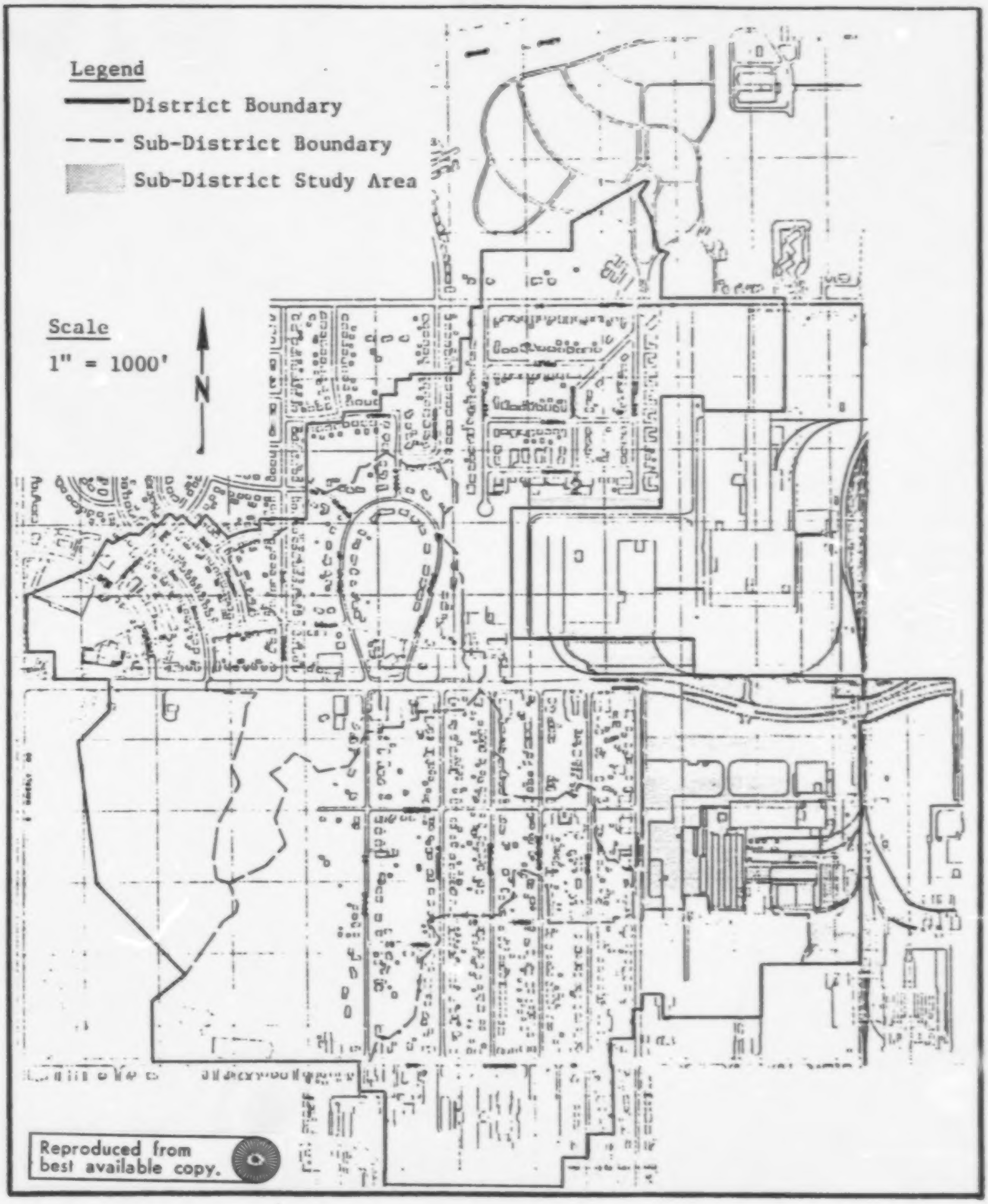
Legend
District Boundary
Sub-District Boundary
Sub-District Study Area
Scale
1" = 1000'
N

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FIGURE III-33 SUB-DISTRICT STUDY AREA MONITORED AT STATION 2

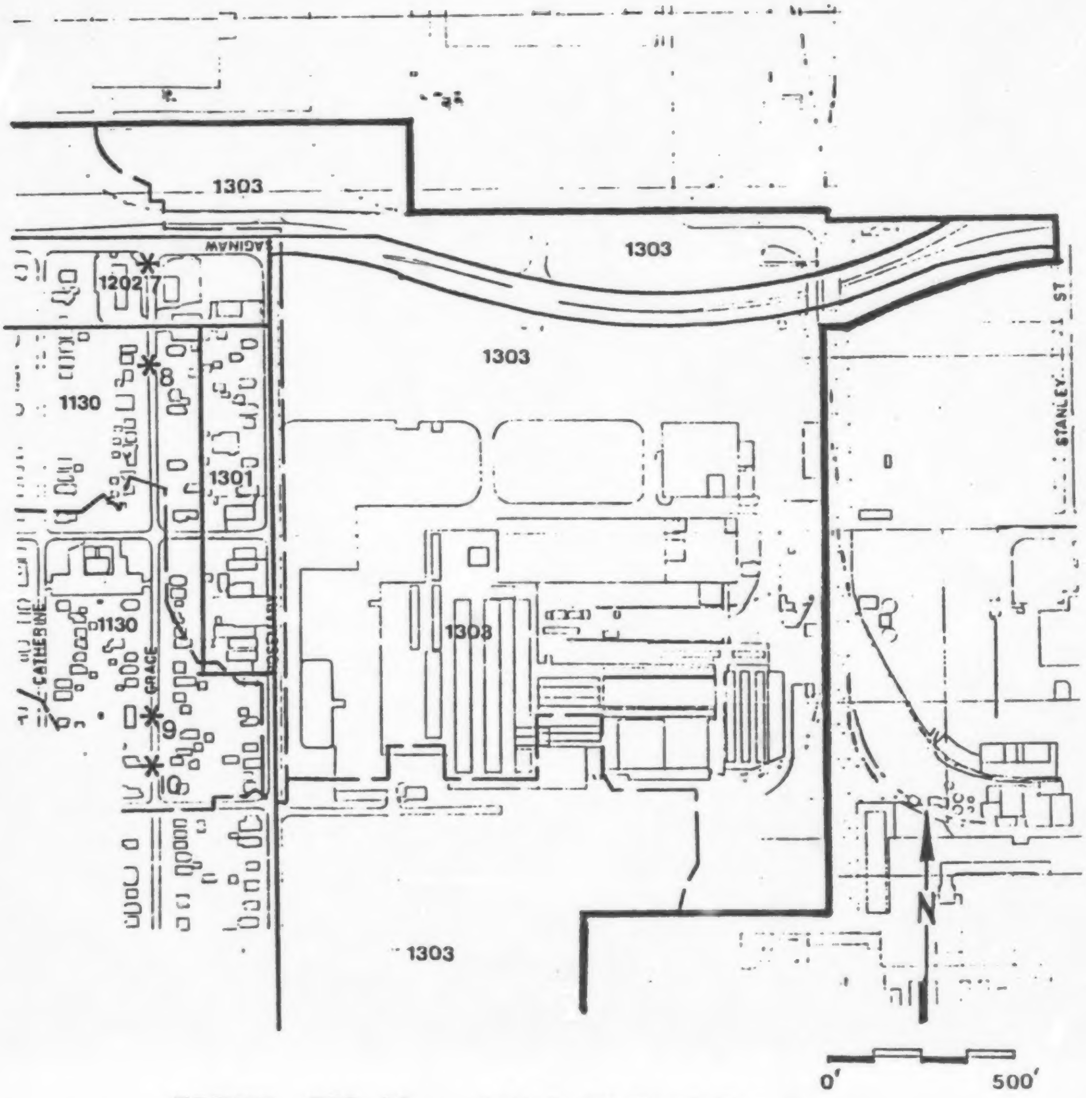

1303
SAGINAW
1303
1202
7
1303
8
1130
STANLEY ST
1301
CATHERINE
1130
GRACE
1308
9
10
N
1303
0'
500'

FIGURE III-34 Level IV Land Use Map Sub-district Monitored at Station 2

TABLE III-14

Features of the Sub-District Study Area
Monitored at Station 2
(Industrial Area)

	Land Use	Cumulative Area (acres) Area	Impervious Area	Effective Impervious Area
1120	Residential (.5 - 2 units/acre)			
1130	Residential (2.5 - 8 units/acre)			
1140	Residential (8 units/acre)			
1202	Commercial (Linear strip)			
1203	Commercial (Shopping center)			
1301	Industrial (light)			
1303	Industrial (heavy)	63.0	40.4	40.4
1400	Parkland			
1401	Institutional			
	Total	63.0	40.4	40.4

Catchment Population	0
Catchment Slope (ft./mi.)	132
Channel Slope (ft./mi.)	31
Length of Channel (ft.)	9,450

District Study Area Monitored at Station 1

As mentioned earlier, Station 1 is the outlet for the entire Bogus Swamp Drainage District study areas as shown in Figure III-35. This area includes all of the areas mentioned earlier for Stations 2-10 plus an additional 44.3 acres of low density housing, 15.3 acres of medium density housing, 9.1 acres of high density housing. 4.2 acres of commercial strip developments, 3.3 acres of light industry, 3.8 acres of heavy industry, 8.6 acres of parkland and 9.5 acres of institutional land uses as presented in Table III-15 and Figure III-36.

Figure III-37 summarizes the land uses for the entire drainage district using Level IV codes on a Level III map scale. In addition it should be noted again that a very detailed list of fixed site data for each sub-district is available in the Separate Data Report that was prepared for this project.

Public Works Practices

Traditional practices in the design of storm drainage facilities include using the so-called 10-year frequency event as the design storm for calculating needed hydraulic capacity. Runoff volumes are calculated using the rational formula.

New subdivisions require curbs and gutters, inlets, and enclosed tile for storm drainage. Building code requirements call for downspouts to be to the surface of the lot.

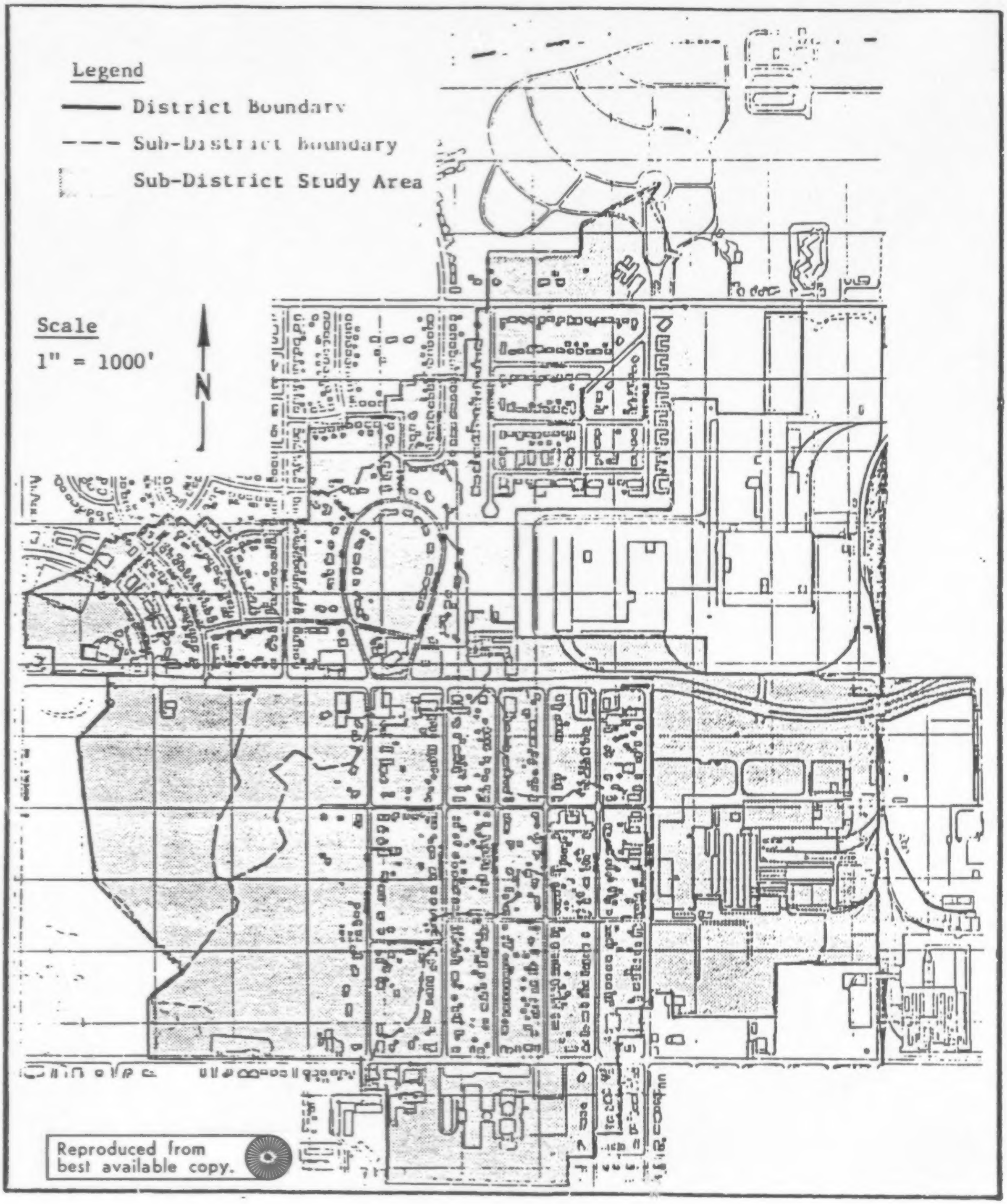

Legend
District Boundary
Sub-District Boundary
Sub-District Study Area
Scale
1" = 1000'
N
Reproduced from best available copy.

FIGURE III-35 DISTRICT STUDY AREA MONITORED AT STATION 1

Reproduced from best available copy.

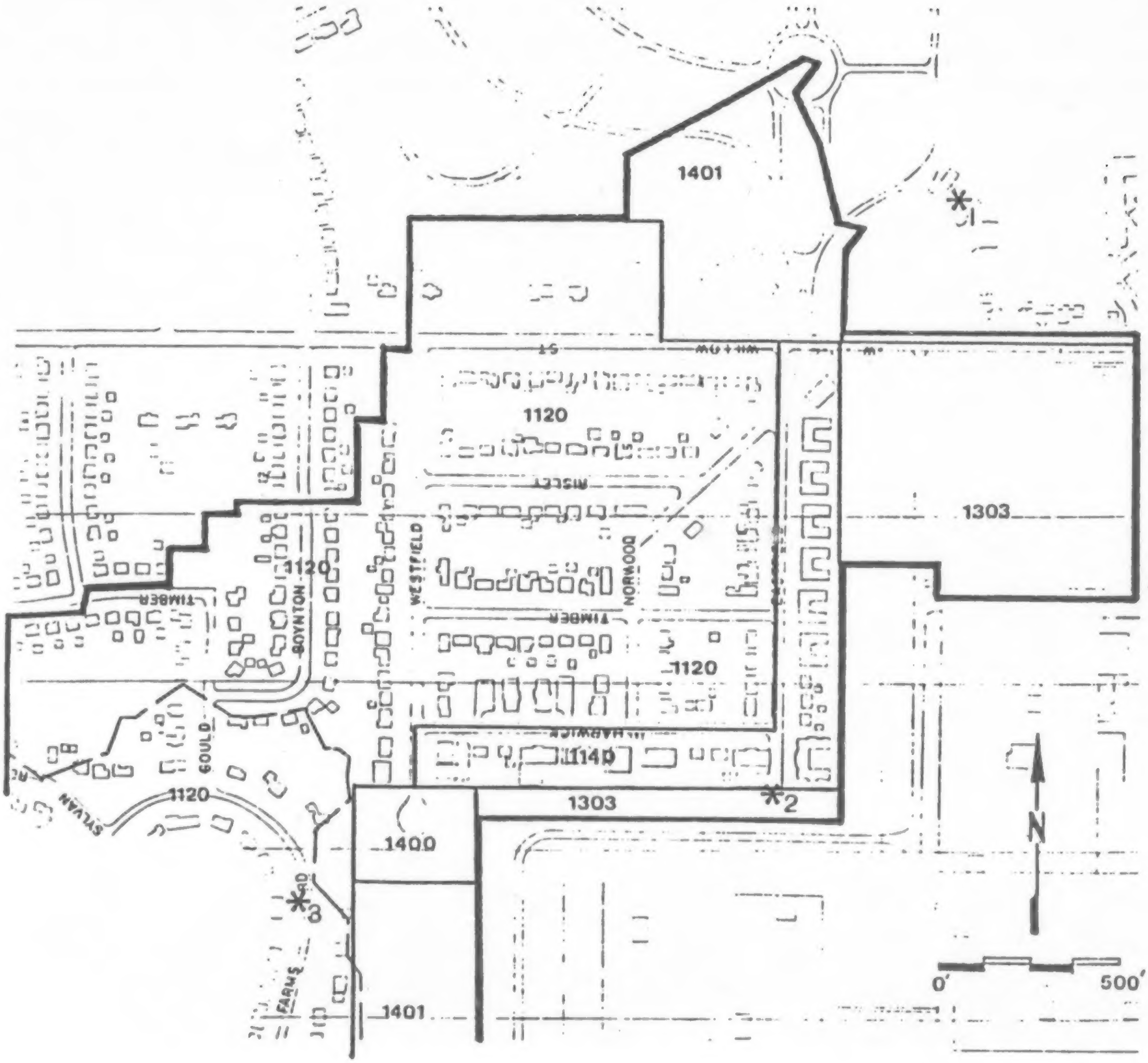

1401
1120
1303
1120
1120
1120
1140
1303
1400
1401
TIMBER
TIMBER
TIMBER
BOYNTON
WESTFIELD
NORWOOD
GOULD
SYLVAN
FARMS
HARWICK
ST
*2
*3
N
0'
500'

FIGURE III-36 Level IV Land Use Map Sub-district Monitored at Station 1

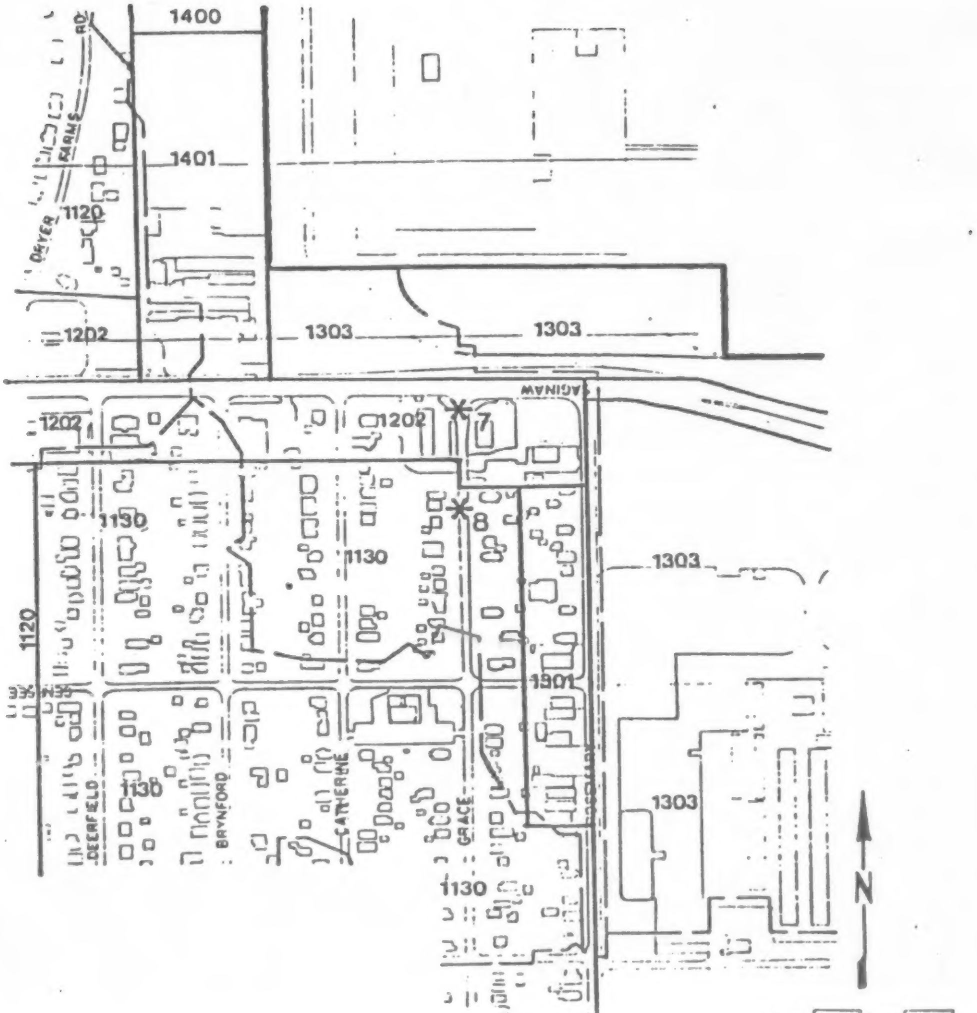

1400
RD
FARMS
DWYER
1401
1120
1202
1303
1303
SAGINAW
1202
1202
7
8
1130
1130
1303
1120
GENESEE
1301
1130
DEERFELD
BRYNFORD
CATHERINE
GRACE
1303
1130
N
0'
500'

FIGURE III-36 (Contin.) Level IV Land Use Map Sub-district Monitored at Station 1

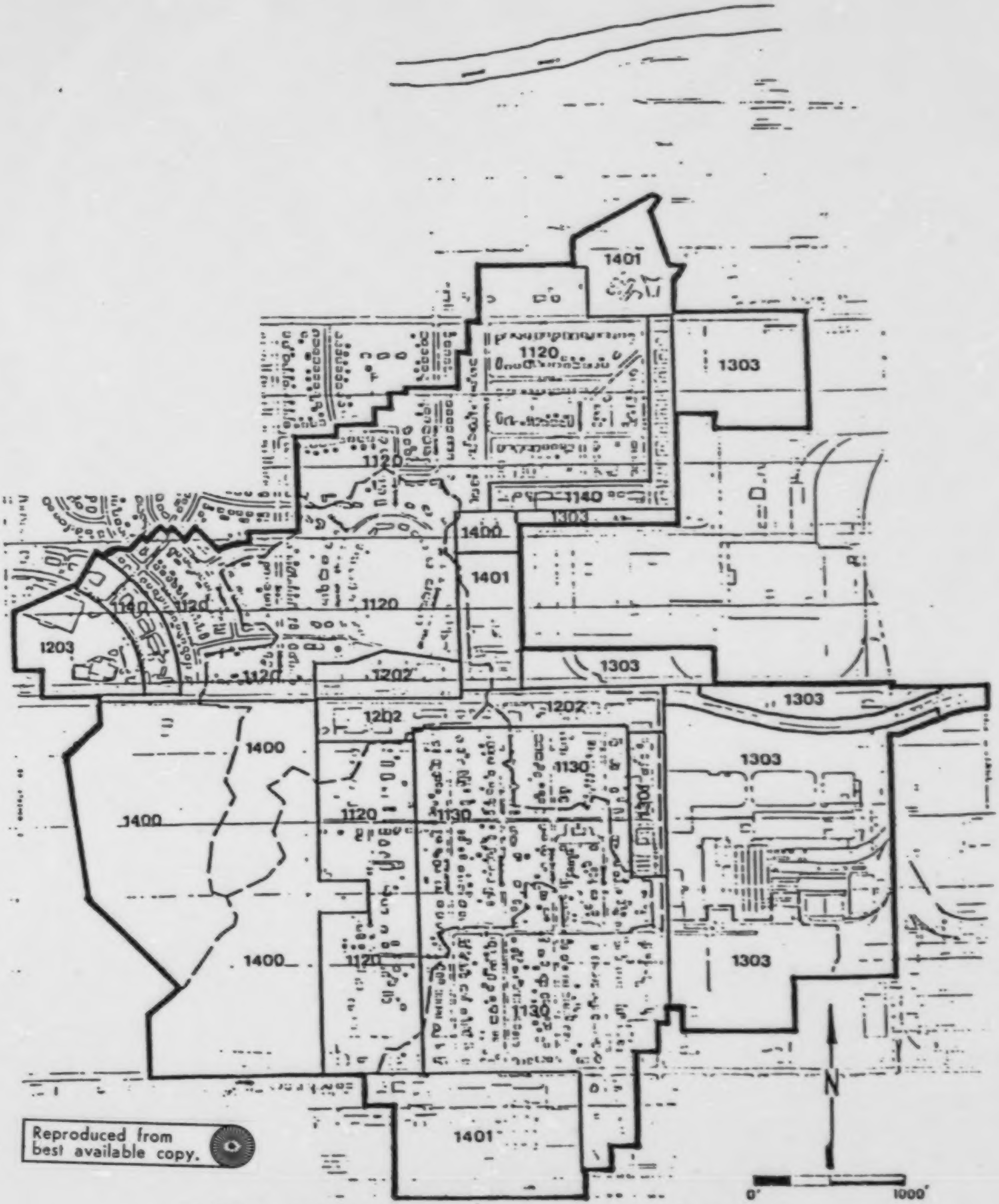
TABLE III-15

Features of the Sub-District Study Area Monitored at Stations 1 (Study Area Outlet)

		Cumulative Area (area)		
	Land Use	Area	Impervious Area	Effective Impervious Area
1120	Residential (.5 - 2 units/acre)	126.5	37.4	24.5
1130	Residential (2.5 - 8 units/acre)	76.9	28.2	18.5
1140	Residential (8 units/acre)	14.3	8.3	6.5
1202	Commercial (Linear strip)	13.2	9.7	9.7
1203	Commercial (Shopping center)	10.1	10.1	10.1
1301	Industrial (light)	3.3	2.2	2.2
1303	Industrial (heavy)	83.2	52	52
1400	Parkland	91	5.6	5.6
1401	Institutional	34.1	12.5	12.5
	Total	452.6	166	141.6

Catchment Population	2,250
Catchment Slope (ft./mi.)	221
Channel Slope (ft./mi.)	32
Length of Channel (ft.)	49.500

92



1401
1120
1303
1120
1140
1303
1400
1401
1120
1140
1120
1203
1120
1202
1303
1303
1202
1202
1400
1400
1130
1303
1120
1130
1303
1400
1120
1303
1130
N
1401
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0'
1000'

FIGURE III-37 Summary of District Land Use

Street sweeping is conducted by the Ingham County Road Commission and occurs on primary roads every 2-3 weeks. Secondary and residential streets in subdivisions are swept 2-3 times a year. The County Road Commission also has responsibility for winter road maintenance and salt is applied to primary roads as needed. Secondary and subdivision roads are plowed with the exception of a few major intersections and steep grades that may be salted when snow and/or ice conditions require. Weekly solid waste collection occurs on a contract basis between a private-sector vendor and individual property owners. Studded tires have been prohibited in Michigan for two years. Catch basins are maintained by the County Road Commission and are cleaned two or three times a year.

In drainage districts being planned now by the Ingham County Drain Commissioner there is a genuine effort made to incorporate stormwater retention, detention, and infiltration in such projects.

BMP DESCRIPTION

In-line Wet Retention Basin

The in-line wet retention basin is located on the Waverly Hills Golf Course in the Southeast corner of the Bogus Swamp Drainage District. The location of the basin, determined from a U.S.G.S. map, is longitude 84° 26' 00" and latitude 42° 44' 20". The area in which the basin is located was originally a lowland area which retained drainage from the surrounding golf course land. The bsain was formed by increasing the volume, both by increasing depth and surface area through removal of natural material and contouring the basin side. Drainage from upstream commercial and residential areas was then routed to the basin through a new netweork of storm sewers. Thus, the current retention basin receives both the original drainage from a portion of the golf course and the rerouted flow from the additional upstream area.

The main design goal for this basin was to provide flood relief for downstream areas by reducing peak flow rates during large storm events. A second goal was to provide control of the water quality of the runoff water discharged from the basin. This goal was incorporated into the design by providing a highly restrictive maximum outlet flow, thus increasing the retention time in the basin. The relatively large capacity available in this basin in relationship to the drainage area allowed for the extra capacity required beyond normal flood control requirements. A third objective of the design was to provide the possibility ot a water supply for irrigation requirements of the golf course. The golf course management expressed a desire to utilize the water in the retention basin for irrigation provided the water quality was acceptable for this purpose. Assuming that irrigation is possible, a further benefit in terms of water quality control will occur through an actual reduction in water volume discharge (i.e. pounds of pollutant) and through increased retention time in the basin during storm events.

Figure III-38 shows the general basin geometry. Inlet water to the basin from the commercial and residential ares is routed via 410 feet of 36 inch pipe. The inlet enters the basin below the standing water elevation. The basin is designed to maintain a standing water elevation of 848.5 with a maximum flood stage of 852.0. Design is based on a maximum stage of 851.3 during the ten year storm. Depth-volume and depth-surface area relationships are presented in Table III-16.

The discharge rate is controlled by the outlet structure and downstream pipe. The pond outlet structure consists of a standard pipe structure made of precast concrete with a Neenah-type gate inlet. as shown in Figure III-39. The structure is four feet in diameter and four feet deep. An eight inch diameter shear gate at the invert elevation provides the ability to lower the water elevation for maintenance or dredging purposes. A circular baffle is installed around the outlet to force a downward movement of water prior to discharge through the outlet. The baffle is present to minimize surface effects in the pond, minimize short circuiting, and to prevent discharge of floating materials during average storm events. Effluent from the basin is conveyed back to the main drain via 943 feet of 12 inch pipe on a relatively flat grade (0.3%). The combination of the relatively small, flat-sloped outlet pipe and the outlet structure results is a highly restricted outlet flow rate which changes only slightly under varying heads in the pond. The maximum discharge rate under gravity flow conditions is 1.8 cfs and is 3.0 cfs under pressure

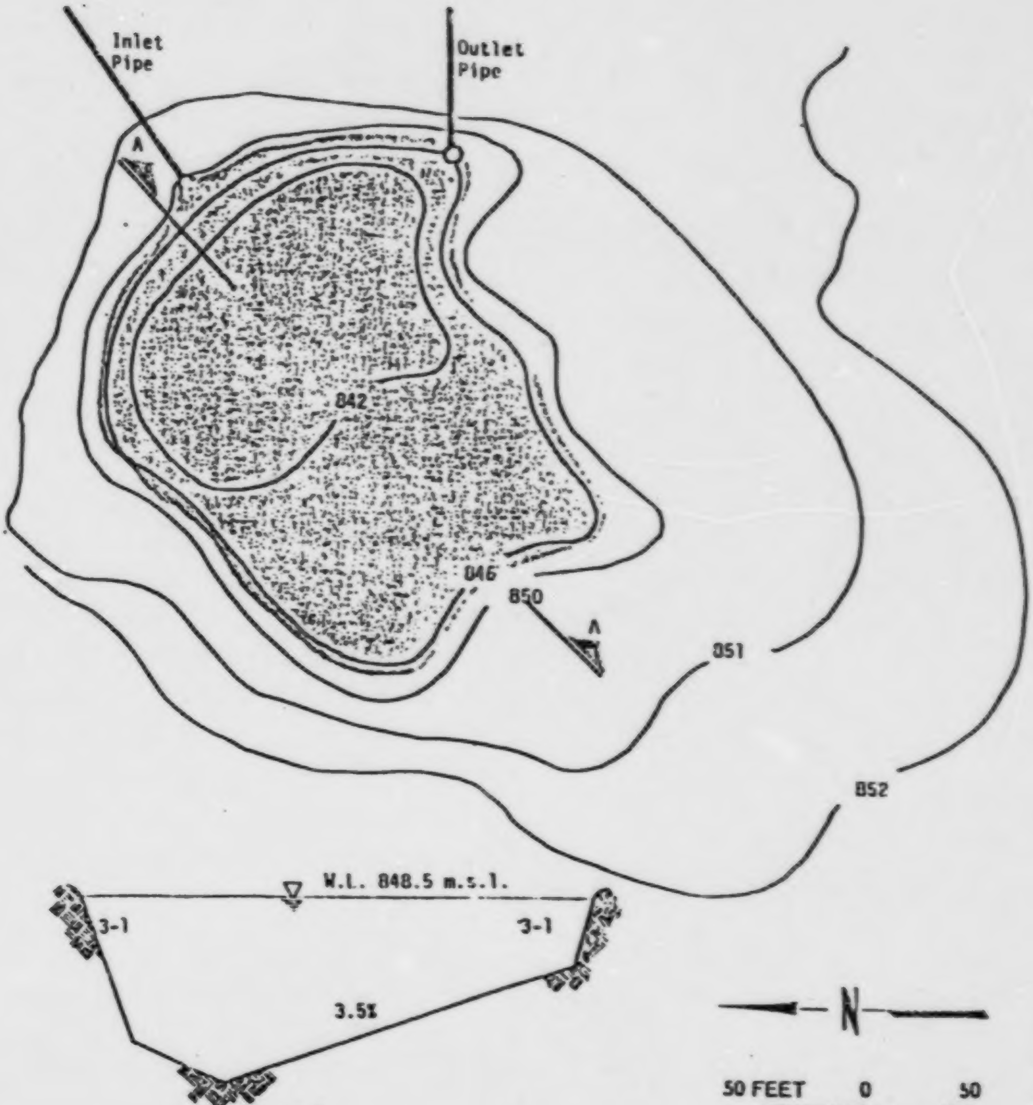
Inlet Pipe
Outlet Pipe
A
842
846
850
A
851
852
W.L. 848.5 m.s.l.
3-1
3-1
3.5%
Section A-A
N
50 FEET 0 50

FIGURE III-38 In-line Retention Basin BMP

97

TABLE III-16

Golf Course Retention Basin Depth-Volume and Depth Area Relationship

Stage ft.	Volume ft.^{3}	Surface Area ft.^{2}
848.5	233.000	49,000
894.0	258,000	50,700
850.0	311,000	54,200
851.0	390,000	107.000
852.0	543,000	197.000

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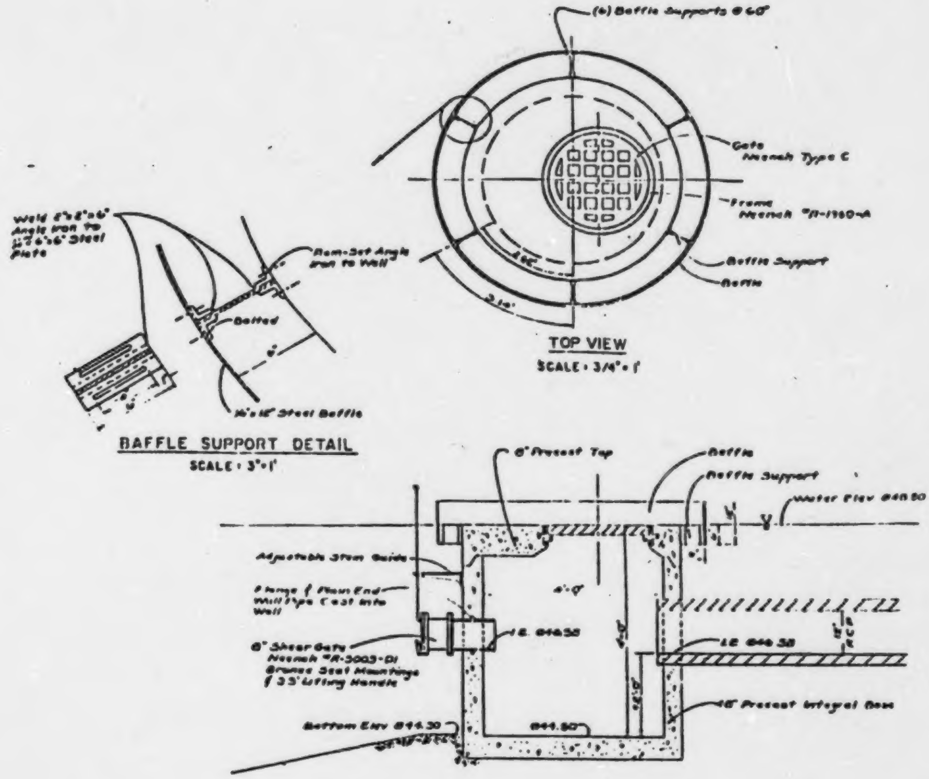
(6) Baffle Supports @ 60°
Gate Neenah Type C
Frame Neenah "R-1960-A
Baffle Support
Baffle
3'-6"
TOP VIEW
SCALE: 3/4"=1'
Weld 2"x2"x¼" Angle Iron to ½"x6"x6" Steel Plate
Ram-Set Angle Iron to Wall"
Bolted
¼"x16" Steel Baffle
BAFFLE SUPPORT DETAIL
SCALE: 3"=1'
6" Precast Top
Baffle
Baffle Support
Water Elev 848.50
Adjustable Stem Guide
Flange & Plain End Wall Pipe Cast into Wall
6" Shear Gate Neenah "R-5003-D1 Bronze Seat Mountings & 3.5' Lifting Handle"
I.E. 846.50
4'-0"
I.E. 846.50
16" Precast Integral Base
Bottom Elev 844.30
844.50

FIGURE III-39 Outlet Structure - In-line Retention BMP

conditions at an elevation of 851.3 (2.8 ft. head, 10 year storm). As can be seen, the outlet flow remains fairly constant during a wide range of storm sizes due to the configuration of the outlet.

Upsized Pipes

Two upsized pipe sections have been installed under Grace Street, between Saginaw and Michigan Avenues, in the southern portion of the Bogus Swamp Drainage District. The approximate location of these two pipe sections as determined from a U.S.G.S. map is longitude 84° 35' 20" and latitude 42° 44' 20". The pipes are installed in searies with the downstream pipe receiving additional urban runoff in addition to the effluent from the upstream pipe.

The basic design concept of the upsized pipes was to provide a location to allow for sedimentation of suspended material to occur. The primary benefit anticipated was the control of urban runoff water quality through the ulse of the sedimentation process. Water quality improvement was anticipated to include reduction of nutrients and metals associated with the suspended solids. The upsized pipes were designed to be sediment traps by slowing the velocity of the water in the drainage system and providing a sump for the accumulation of settled material. Accumulated material could then be removed by periodic pumping of the sludge material from the pipes.

Figures III-40 and III-41 show the physical dimensions of the two pipes. The pipes are designed so that the crown elevation of the inlet and outlet pipes are the same as the crown elevation of the upsized pipes at the respective inlets and outlets. Manholes are locatded at the entrance and exit locations to provide access to the pipes for maintenance purposes and provide the transition element from the smaller pipes to the large pipe. This design provides space for sediment collection and controls the cross sectional area of the pipe through which the storm drainage must flow. The design provides for retention of small storm events which are numerous in this area, as well as many other areas of the country.

The upstream BMP (south pipe) is a 96 inch diameter reinforced concrete pipe (R.C.P., C-76, CL-III) with a length of 144 feet. The outlet pipe from this BMP is 54 inches in diameter providing a standing water depth of 42 inches in the BMP. The downstream BMP (north pipe) is a 96 inch diameter reinforced concrete pipe (R.C.P., C-76, CL-III) with a length of 85 feet. The outlet pipe from this downstream BMP is 60 inches in diameter providing a standing water depth of 36 inches in the BMP. Following installation, a weir plate was placed at the outlet of the north pipe for flow measuring purposes. This resulted in a standing water depth of 42 inches during the study.

Table III- 17 shows estimated design conditions for one year and six month storms developed from synthetic design hyetographs for southern Michigan. Design flow conditions

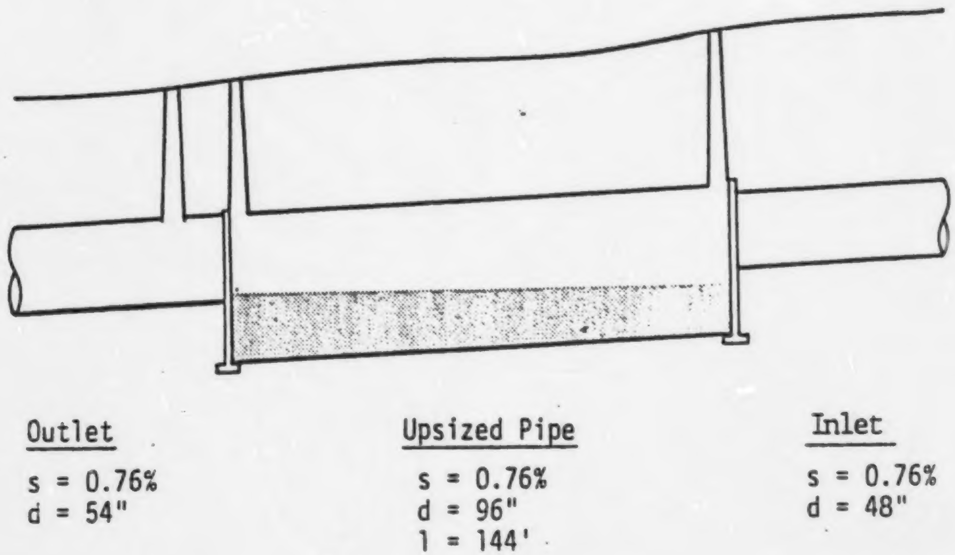
Outlet
s = 0.76%
d = 54"
Upsized Pipe
s = 0.76%
d = 96"
l = 144'
Inlet
s = 0.76%
d = 48"

FIGURE III-40 Upstream Upsize Pipe BMP

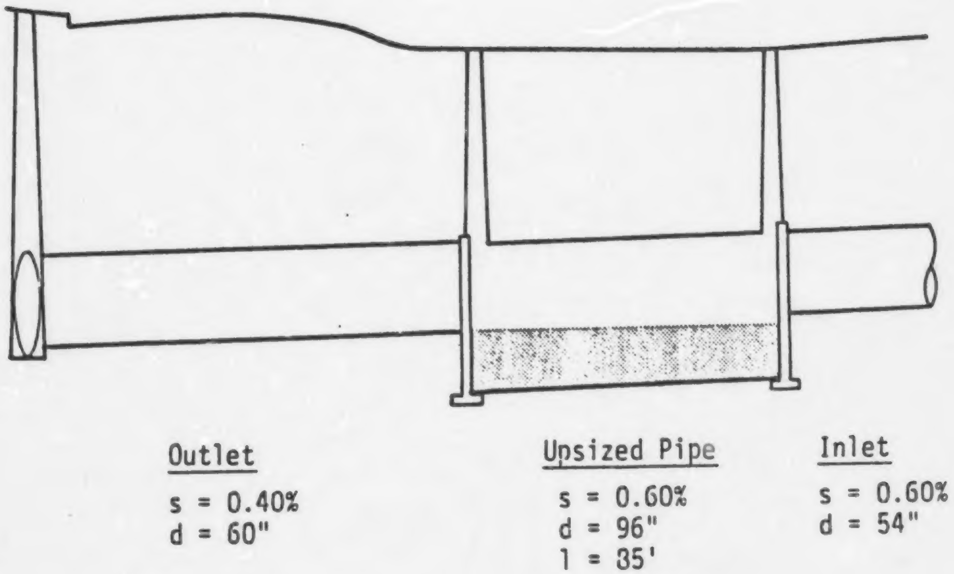
Outlet
s = 0.40%
d = 60"
Upsized Pipe
s = 0.60%
d = 96"
l = 35'
Inlet
s = 0.60%
d = 54"

FIGURE III-41 Downstream Upsize Pipe BMP

TABLE III- 17

Upsized Pipes
Initial Design Criteria

	Flow (cfs)		Velocity (fts/sec)		Detention Time (minutes)	
Storm Frequency	Peak.	Ave.	Peak.	Ave.	Min.	Ave.
Upstream pipe						
1 yr. storm	22	3.2	0.70	0.12	3.4	20
6 month storm	11	2.5	0.38	0.10	6.3	24
3 month storm	9.5	2.1	0.33	0.08	7.3	30
Downstream pipe						
1 yr. storm	35	5.2	0.85	0.17	1.7	8.3
6 month storm	18	4.0	0.49	0.13	2.9	10.9
3 month storm	15	3.5	0.42	0.12	3.4	11.8

for various design storms were estimated using the drainage area characteristics and design hyetographs. At the south upsized pipe, the estimated peak flow will result in a peak velocity of 0.70 feet per second for the one year storm, and 0.38 feet per second for the six month storm. The north upsized pipe will experience velocities of 0.85 feet per second at the peak of the one year storm and 0.49 feet per second at the peak of the six month storm. The earlier mentioned concept for these upsized pipes was to provide for the control of suspended solids and associated pollutants for the many small storms which occur during the year. Storms smaller than the six-month or one-year design event will be slowed more and retained lnger. Thus, the upsized pipes have been designed to provide conditions favorable to sedimentation for storms which occur frequently during the year.

Larger and more intense storms will result in increased peak flows and a deterioration of performance. During high peak flow periods scour may begin to occur and previously deposited materials could be flushed out. Therefore, solids removal from the basin on a periodic basis will be very important to assure large deposits of material are not available for scour during large storms.

It is expected that a key element of the maintenance program for these pipes will be the removal of the accumulated solids from the pipe. Solids must be removed periodically so that sufficient capacity is available in the sump area to trap material and so that trapped materal is not scoured from the pipe during large storms.

Off-line Detention Basin

The off-line dry detention basin is located in an area of the Bogus Swamp Drain commonly referred to as Dryer Farms. The approximate location of this basin as determined from U.S.G.S. maps is at longitude 84° 35' 40" and latitude 42° 44' 35". Both the inlet and outlet of this basin are being monitored during the study to obtain data for performance evaluation.

The area in which the basin is located is a natural lowland area behind single-family homes. No actual construction activities were required in this area as the natural land contours are being utilized for detention purposes. The basin area is grassed and is an extension of the residential area (37.3 acres) and some commercial (4.0 acres) and public (1.3 acres) land. In addition, the discharge water from the on-line retention basin contributes to the flow, as it is upstream on the same branch of the drain.

The main design objective of this detention area was as a flood relief point within the drainage district. Actually, the reason this area periodically floods is due to the limited downstream capacity of the storm drain system. An original design alternative was to increse the carrying capacity of the drain in this area by installing larger pipes downstream. However, the residents of the area indicated that occassionaly flooding in the low lying area was not a major concern. Therefore, the existing pipe in

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this area was simply cleaned during the construction project, and the detention basin was in essence established. The second objective of this detention basin was to provide some control of water quality during detention periods. Water qualiy improvement could occur duirng large storms due to sedimentation and infiltration in the the detention area.

Figures III-42 and III-43 show the general area and topography of the basin. A 36 inch concrete pipe carries water into and through the basin. The capacity of this pipe is approximately 20 cfs. Capacity is limited in the section of pipe running through the bottom of the basin due to a flat slope. The actual slope of the pipe is unknown due to extensive settling which has occurred along this length of pipe. A manhole with a beehive gate is located just upstream of the restricted flow section. This opening is at an elevation of 8390.

During average storm events with flows up to a peak of 20 cfs, water passes directly through the conduit system. When the inlet rate exceeds 20 cfs, water backs up to the beehive structure and flows into the dry basin. Flow will continue to enter the basin until the inlet rate drops below the downstream pipe capacity. At this point. water will begin to discharge from the detention area until the basin is empty. Depth-volume and depth surface area relationships for this material detention area are shown in Table III- 18

The original design estimates indicated that flooding in the basin would occur a few times each year. Given the

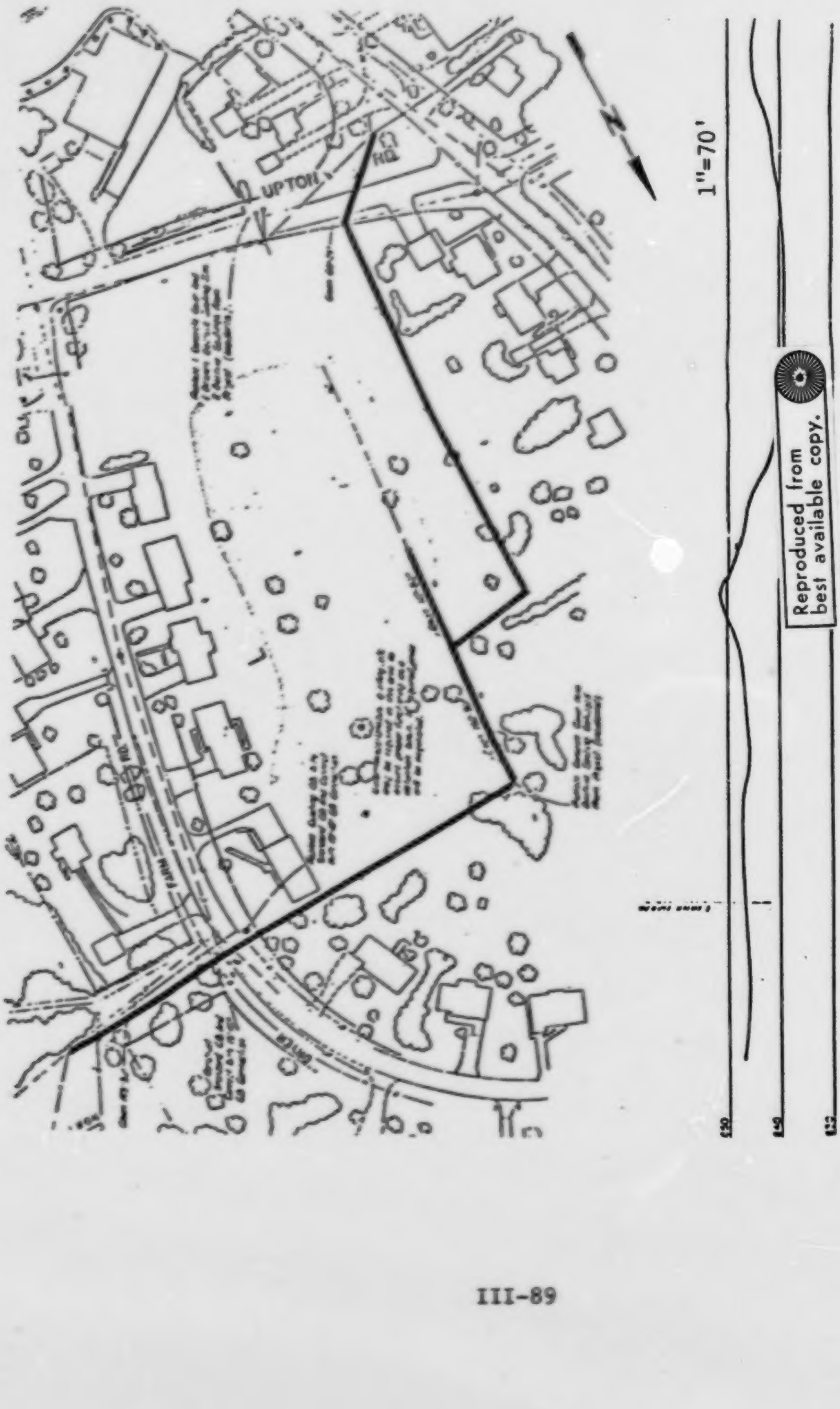
UPTON
RD
FARM RD
N
1"=70'

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FIGURE III-42 Off-line Detention Basin General Area

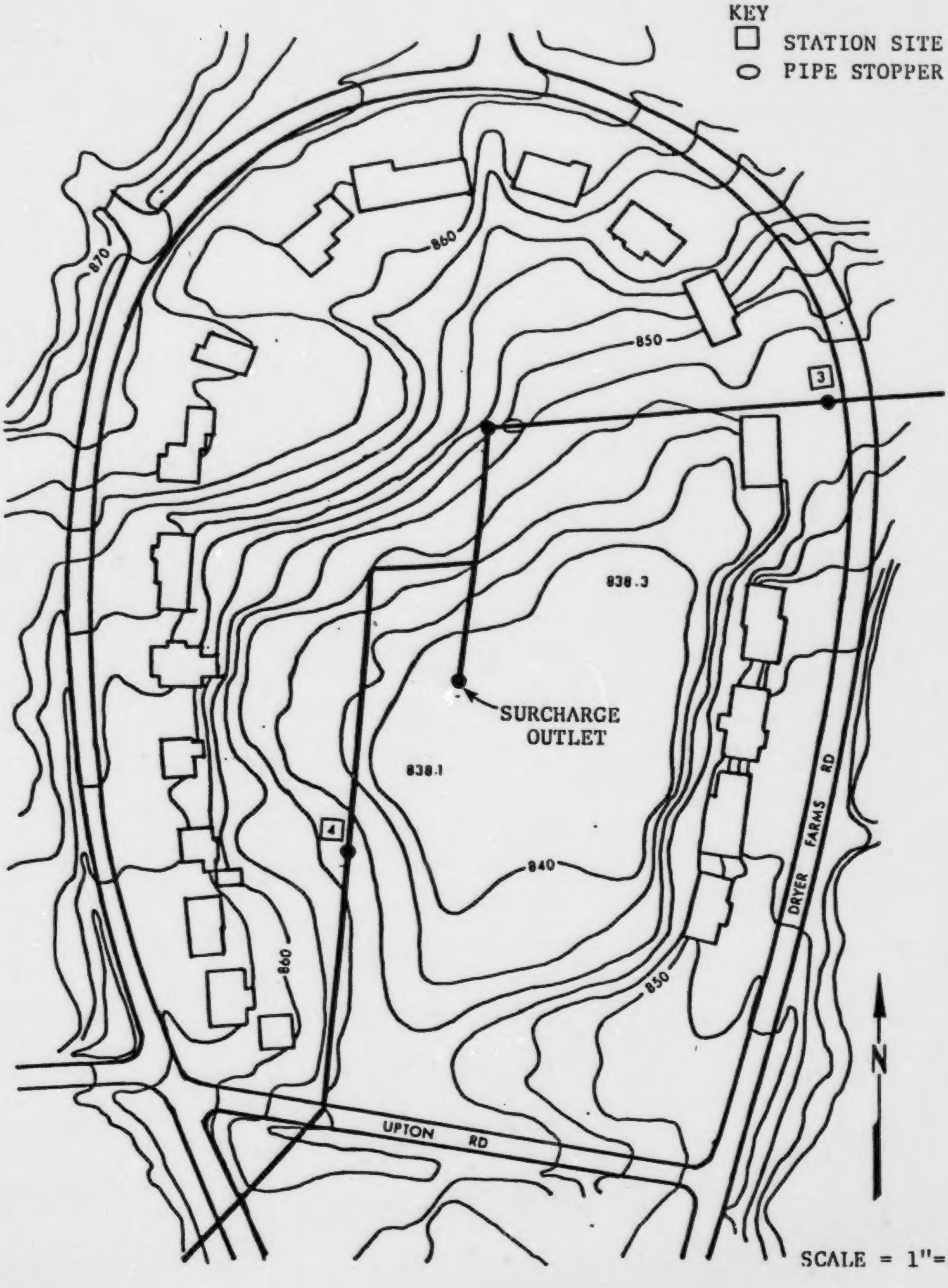

KEY
☐ STATION SITE
○ PIPE STOPPER
870
860
850
3
838.3
SURCHARGE OUTLET
838.1
4
840
860
850
DRYER FARMS RD
UPTON RD
N
SCALE = 1"=139'

FIGURE III-43 Off-line Detention Basin Topography

TABLE III-18

Off-line Dry Detention Basin
Depth-Volume, Depth-Area Relationship

Storage elevation	feet	Capacity ft^3	Surface Area ft^2
839	0	0	23,400
841	2	89,525	66,125
843	4	256,850	101,250
845	6	488,050	129,950
846	7	632,563	159,075
847	8	806.538	188,875

uncertainty of the extent and frequency of flooding in the area, it was decided to intentionally induce flooding in the basin for investigative purposes. An inflatable pipe stopper with a small PVC pipe under the pillow was installed in the downstream line to further restrict flow on a controlled basis. Initially, a target flow rate of 8 cfs was used for the restricted outlet capacity. However, based on actually measured flows during storm events, it was necessary to restrict the maximum rate to 4 cfs in order to induce any significant flooding in the basin.

SECTION IV.

DATA COLLECTION PROGRAM METHODOLOGY

INTRODUCTION

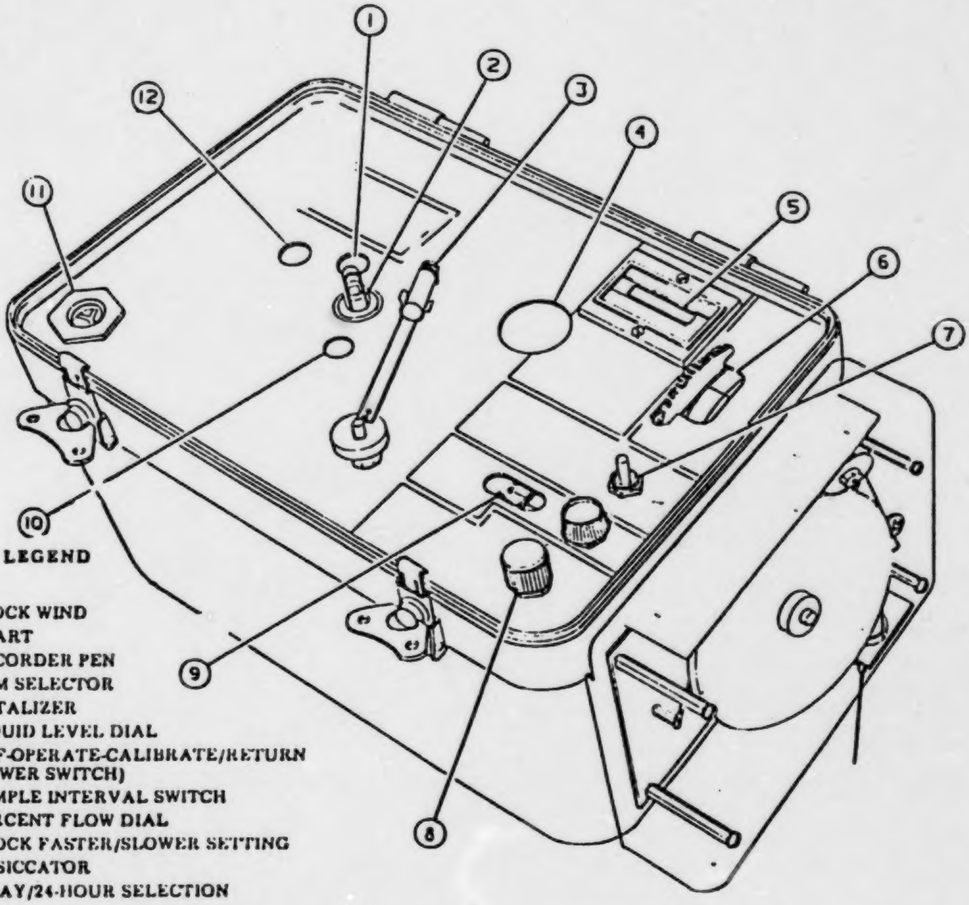
A major portion of this study focused on the collection and analysis of water quality samples. The need for detailed data on the pollutant levels associated with urban runoff and on the ability of various BMP's to remove these pollutants was a major reason for the implementation of the NURP program. A detailed data acquisition plan was developed to obtain this type of information. This plan included the selection of sampling equipment, selection of stations and station design, determination of sampling and analytical protocol, adherence to a quality assurance plan and management of the data. This chapter presents a description of these aspects of the study which served as a basis for the evaluations presented in subsequent chapters.

EQUIPMENT AND INSTRUMENTATION

Flow Meters

Dipper and sonar type flow meters, manufactured by Manning Environmental Corporation, and two bubbler type flow meters, manufactured by Instrument Specialties Co. (ISCO) and Marsh McBirney, were used in this study. The specific models used are as follows:

UF 1100 Ultrasonic	(Manning)



LEGEND

1. CLOCK WIND
2. CHART
3. RECORDER PEN
4. CAM SELECTOR
5. TOTALIZER
6. LIQUID LEVEL DIAL
7. OFF-OPERATE-CALIBRATE/RETURN (POWER SWITCH)
8. SAMPLE INTERVAL SWITCH
9. PERCENT FLOW DIAL
10. CLOCK FASTER/SLOWER SETTING
11. DESICCATOR
12. 7-DAY/24-HOUR SELECTION

Figure IV - 1 Dipper Unit Schematic

F3000A Dipper	(Manning)
Model 265 Bubbler	(Marsh McBirney)
Model 1870 Bubbler	(ISCO)

Each is capable of providing flow rate and volume via a chart and totalizer readout. In addition, the units can be interfaced with samplers to enable the collection of flow proportionate samples automatically.

A program module in the UF1100 and model 1870, and a subassembly cam in the F3000A, convert level measurements to flow by means of flow rating curves developed for flow through and over a variety of structures (i.e., round pipes, flumes, weirs). The dipper unit (Figure IV-1) measures level with an insulated electrical cable that dips into the water and retracts upon contact. This cycle is performed at five second intervals, each time providing level input. The ultrasonic unit has a transducer that transmits an ultrasonic pulse to the water and measures the liquid level by timing the return of the signal. The model 1870 and model 265 bubbler flow meters measure depth by the amount of pressure exerted on an air line outlet. The Model 265 measures velocity in addition to depth, and calculates flow based on multiplying velocity by cross sectional area. Thus, this device when installed and calibrated properly acts similar to a primary flow measuring device (i.e. weir, flume, etc.).

Based on flow determination by these recorders, all of the units can be used to collect flow proportional samples at prescribed intervals when linked to sampling equipment. Each unit tracks total flow by means of a digital totalizer and then pulses the sampler based on throughput volume. All of the units were powered by 12 volt D.C. automobile batteries which eliminated the need for AC power lines. The only modification to the basic units was the addition of large amounts of dessicant held in perforated PVC tubes in the dipper and sonar units. These tubes were placed in the units in the area where the internal battery is normally located and an external battery was used. This modification significantly reduced the moisture related failures of these units which occurred early in the study. Equipment failures were common occurrences during the startup of the program, particularly with the flow recorder units. Further description and solutions to these problems are included in the appendix.

Water Velocity Meter

Water velocity measurements were made using a Marsh-McBirney Model 201 Portable Water Current Meter. This instrument measures flow velocities up to 10 feet per second. It has three full scale ranges: 2.5, 5, and 10 feet per second, any of which can be selected by a rotating switch on the front panel. The accuracy of the instrument is based upon three factors:

1. Long term zero drift which is less than ±0.07 ft./sec.

2. Linearity of response which is ±2% of reading.

3. Absolute calibration which is ±2% of reading.

Water Quality Samplers

Composite and discrete samplers, manufactured by Manning Environmental Corporation, and composite samplers, manufactured by Instrument Specialties Company, were used in this study. A schematic of the Manning unit is shown in Figure IV-2.

The specific models used are as follows:

S-4040 Discrete/Composite	(Manning)
S-3000 Composite	(Manning)
Model 1580 Composite	(ISCO)

The automatic water samplers are capable of collecting variable sample volume (50-500 ml) during each sample collection cycle. Additional volume can be collected using the S-4040 sampler by setting the sampler to cycle 2-10 times consecutively for a single time or flow activation. Both types of sampler are capable of sampling based on a signal from a flow meter or on a present time interval. The S-3000 samplers and ISCO units are composite samplers only and were utilized where flow-proportional composite samples were desired. The S-4040 samplers can be used to collect composite or discrete samples (24, 500 ml bottles per base) by installing the appropriate sample base. The discrete sample collection option provides flexibility in selecting

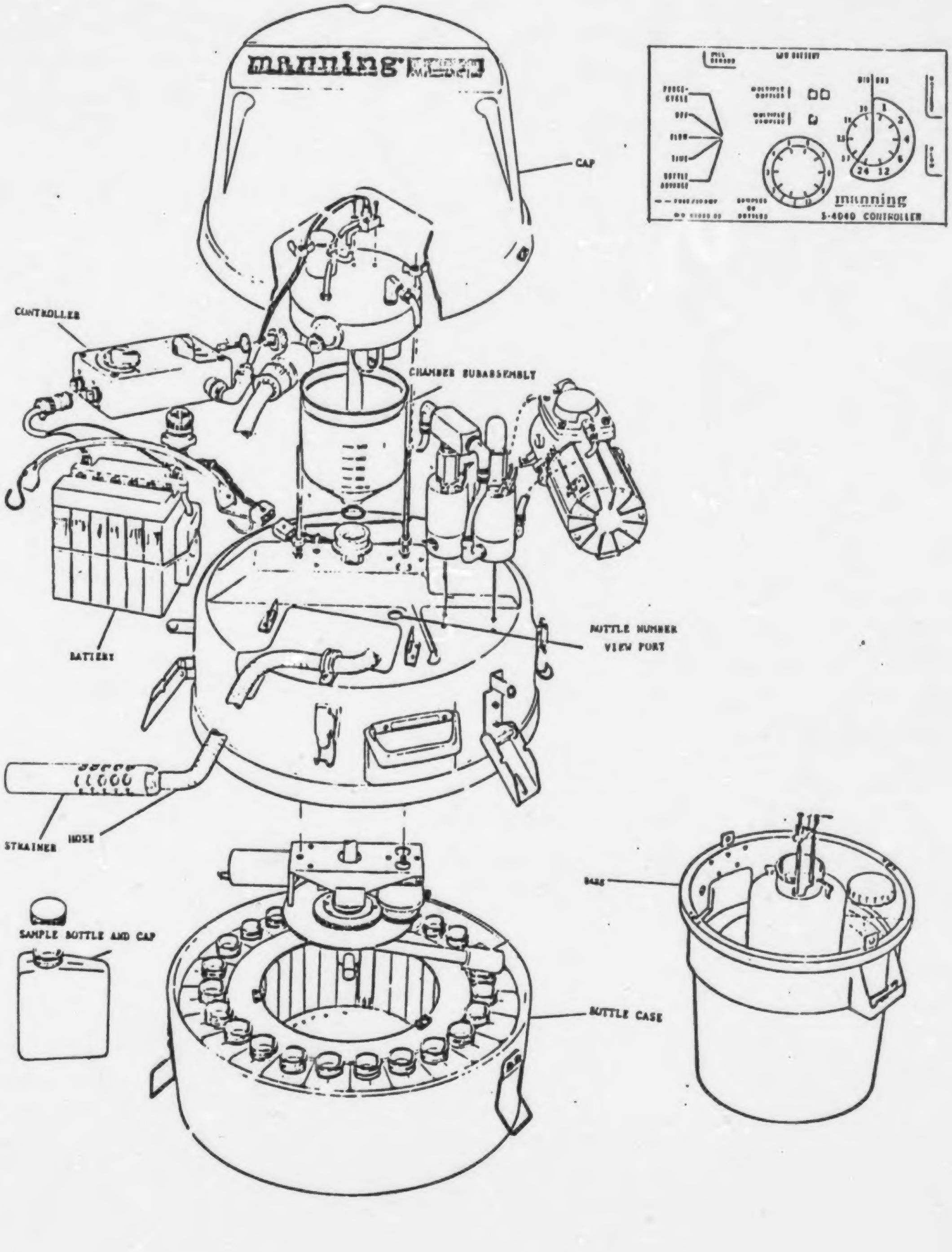

manning
CAP
CONTROLLER
CHAMBER SUBASSEMBLY
BATTERY
BOTTLE NUMBER
VIEW PORT
STRAINER
HOSE
SAMPLE BOTTLE AND CAP
BOTTLE CASE
BASE
manning
S-4040 CONTROLLER

Figure IV - 2 Water Quality Sampler Schematic

the most appropriate sampling methodology for a specific station situation. Using the discrete sampling option and interfacing with a flowmeter, samples can be collected on a flow proportional basis and allow for dividing the sample time period into several composite samples for a given storm runoff period. In this case, the results may be a series of flow composites (4-6 samples) defining various portions of the runoff hydrograph. The discrete sampler can also be used to collect individual samples on a time basis at stations where a flow meter can not be used to pace a sampler. This condition generally occurs at a station where flow conditions do not allow for direct correlation between water depth and flow rate. In this case a station specific stage-discharge relationship is developed using manual measurements of flow. Water level measurements are converted to flow using this relationship and discrete samples are manually composited using flow and time data.

The automatic samplers are either suction lift samplers utilizing a vacuum pump to draw the sample (Manning) or peristaltic pump mechanisms (ISCO). Maximum sample lift is 22 feet with these samplers. The vacuum pump supplies air pressure to purge the sample lines before and after each sample is collected, while the peristaltic pump undergoes reverse flow to purge the sample lines. All sampling equipment is battery operated and thus placement is not dependent on the availability of electric power.

The sampling lines being used with the equipment are 0.95 cm (0.375 inch) I.D. reinforced tubing. This size tubing permits the passage of fairly large solid particles without clogging and still maintains sufficient intake velocity in the line. Intake velocity varies according to the lift, ranging from a minimum velocity of 3 ft./sec. at 22 foot lift to 5 ft./sec. at 3 foot lift. Only one of the stations approaches the 22 foot level (station 2) with all others generally 15 feet or less. Maintaining intake line velocity is important in obtaining representative suspended solids samples.

Special sample intake screens were designed for each sampling station. The screens consisted of two hundred 0.95 cm (0.375 inch) holes, and were secured to a small block anchored in the pipe using a Ram Set. The screen openings were upstream of the anchor block and are oriented directly into the flow. This design provided a secure intake point which resisted movement downstream during high flow, exposing sufficient available surface area to minimize clogging by large debris.

A special option with the sampling equipment was the capability of collecting samples in glass bottles and changing the sample lines and collection system to teflon and glass. This optional setup was used for the collection of samples for the pesticide scans as a part of the monitoring program. This special setup (all teflon and glass) was fitted to the sampling equipment prior to those surveys scheduled for the collection of the pesticide samples.

At most of the stations the samplers were activated by means of a float switch assembly. This assembly was built in house and is illustrated in Figure IV-3. The float switch assembly was needed due to the existence of a small baseflow at most stations. The flow meters measuring the baseflow would periodically accumulate sufficient volume on the totalizer to activate the sampler during periods between storm events. Therefore the float switch assemblies were installed to cut the power to the sample. The normal mode of operation was to run the flow meters continuously while the power to the samplers was off. Once the water level began to rise the switch would activate power to the sampler and then pulses from the flow meter would result in the collection of samples.

The relay box was based on a double position double throw Potter and Brunfield relay. This system uses one set of contacts to initiate power to the sampler and one set of contacts to disconnect the power to the relay via the float switch. The input and output are bypassed to ground to prevent the contacts from burning up. A light emitting diode indicated when the box had been activated. A single position single throw switch was then used to reset the unit.

The float switch was a level sensor capable of being set either in a normally open or normally closed position by rotating the float. The actual switch is a magnetic reed (with the magnet in the float). The float switch was mounted into a fiberglass electrical box with a handmade

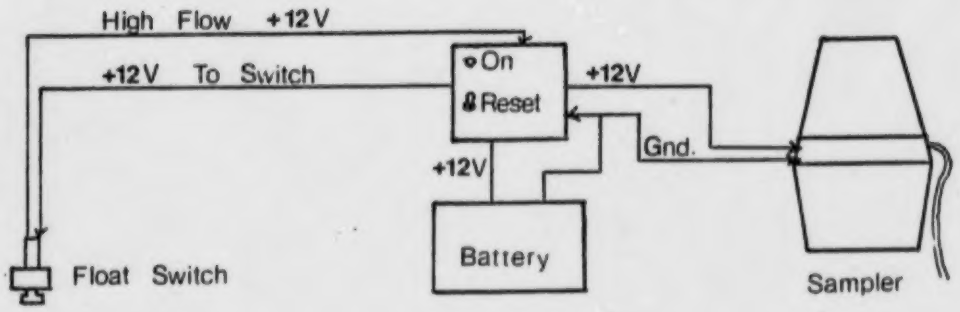

High Flow +12V
+12V To Switch
On
Reset
+12V
Gnd.
+12V
Battery
Float Switch
Sampler

Figure IV - 3 Float Switch Assembly

121

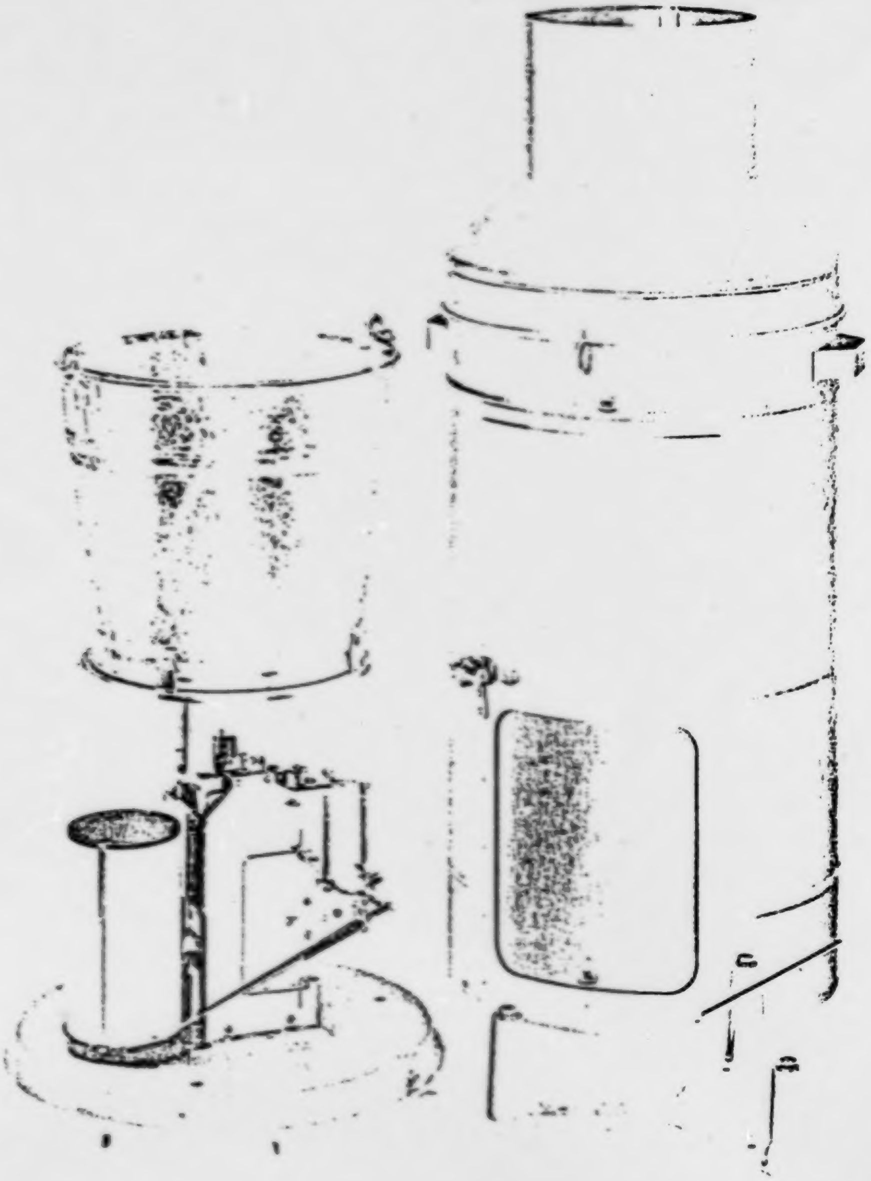


Figure IV - 4 Automatic Rain Gauge

Atmospheric Deposition Samplers

Samples of precipitation (wetfall) for water quality analyses were collected utilizing an automatic rainfall collection unit by Wong Laboratories. These units have a sensor grid which triggers a motor to open the lid when rainfall begins. Rainfall is collected through a 16.5 cm (6.5 inch) diameter funnel draining into a two liter bottle. When rainfall ceases, the sensor grid triggers the lid to close. A special adaptation to this unit was installed to provide for the collection of larger sample volumes during small storms. A separate lid and funnel system which tripled the collection surface area was built and the same sensor was used to trigger both lid assemblies.

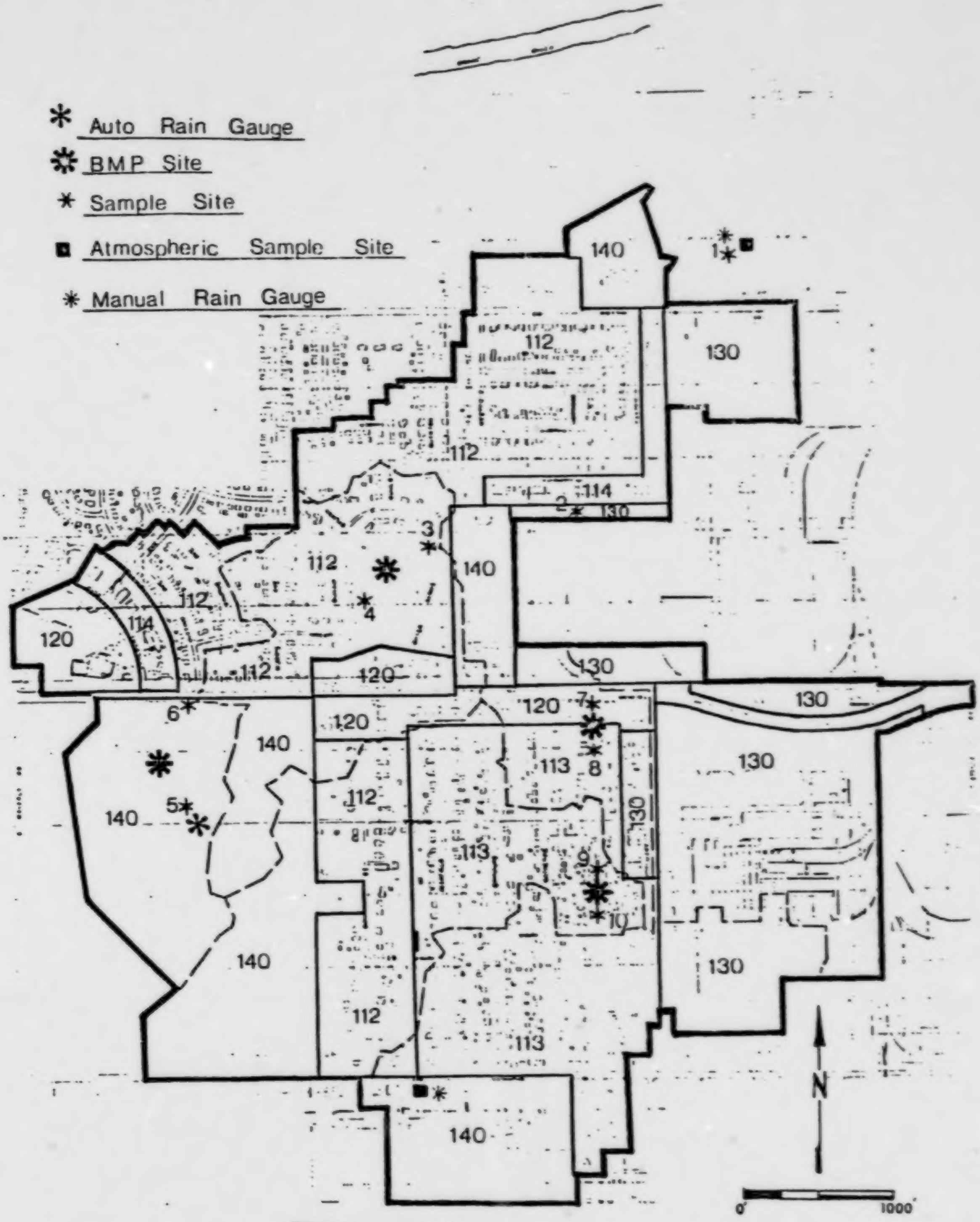
Bulk fallout (dry dust and wetfall combined) was determined using dust buckets open to the atmoshere at all times. These units were built according to ASTM guidelines. Each sampler was a 19.7 cm (7.75 inch) diameter plexiglass cylinder, 23 cm (9 inches) high. An outer cylinder 30 cm (12 inches) in diameter and 33 cm (13 inches) high surrounded the collection vessel to serve as a bird ring. This prevented birds from roosting directly on the collection vessel.

SAMPLING STATION DESCRIPTION

General

Ten water quality stations were maintained within the storm drain system during the project. Stations 1 and 2 served to monitor the study area outlet and an industrial land use area. respectively. The remaining eight stations (3-10) are associated with BMP's and served to identify influent and effluent flows at each of the 4 BMP locations. In addition to stations within the drainage system, water quality in the Grand River was monitored at three different sites. Finally, atmospheric fallout, bulk fallout and wetfall was monitored at two stations. The location of these stations is shown in Figure IV-5.

Station vulnerability to vandalism or flooding and the capability to monitor flow within constraints defined in the Quality Assurance Report are elements which were considered in sampler placement and setup. Two types of installation were used. One utilized utility boxes containing batteries, samplers, and flow meters, with all accessory lines to components (e.g. sensor cables, sample lines, float switches) housed in conduit exiting the box. The conduit in most cases was buried in the ground, and entered the manhole by being wedged between the collar and manhole foundation. Another variation of this type of installation, used with dipper units, required placing the dipper unit on a shelf in the manhole so that the probe could hang vertically, allowing the sensor mechanism to work properly.


Auto Rain Gauge
BMP Site
Sample Site
Atmospheric Sample Site
Manual Rain Gauge
140
1
112
130
114
2
3
4
120
5
6
7
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113
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Figure IV - 5 Station Location

The second type of installation was used in the case of sampling stations where the manhole is located on a paved roadway, or where some other constraint prevented an above ground installation. Such a situation required that all equipment be contained in the manhole itself. Brackets clamped to the manhole strips supported samplers, while flow recorders were placed on shelves secured to the manhole wall. While all dipper probes were hung free, brackets were used to mount the transducers for the sonar units. Signal distortion due to catch basin flows, turbulence, and other influences was minimized by careful selection of bracket location and design. These installations are shown schematically in Figure IV-6.

Station 1 (District Outlet)

This station, located near the outlet of the entire drainage district, represents the flow and quality from 452 acres of land. Flow at this station includes all of the discharge water from the BMP's and some additional uncontrolled runoff water. The sampling station was located at a manhole in the 72 inch main pipe. A 54 inch overflow line runs parallel to the 72 inch pipe and receives water when peak flow exceeds 215 cfs. All equipment, flow recorder, sampler, battery, etc. was housed in a security box above ground, next to the manhole location.

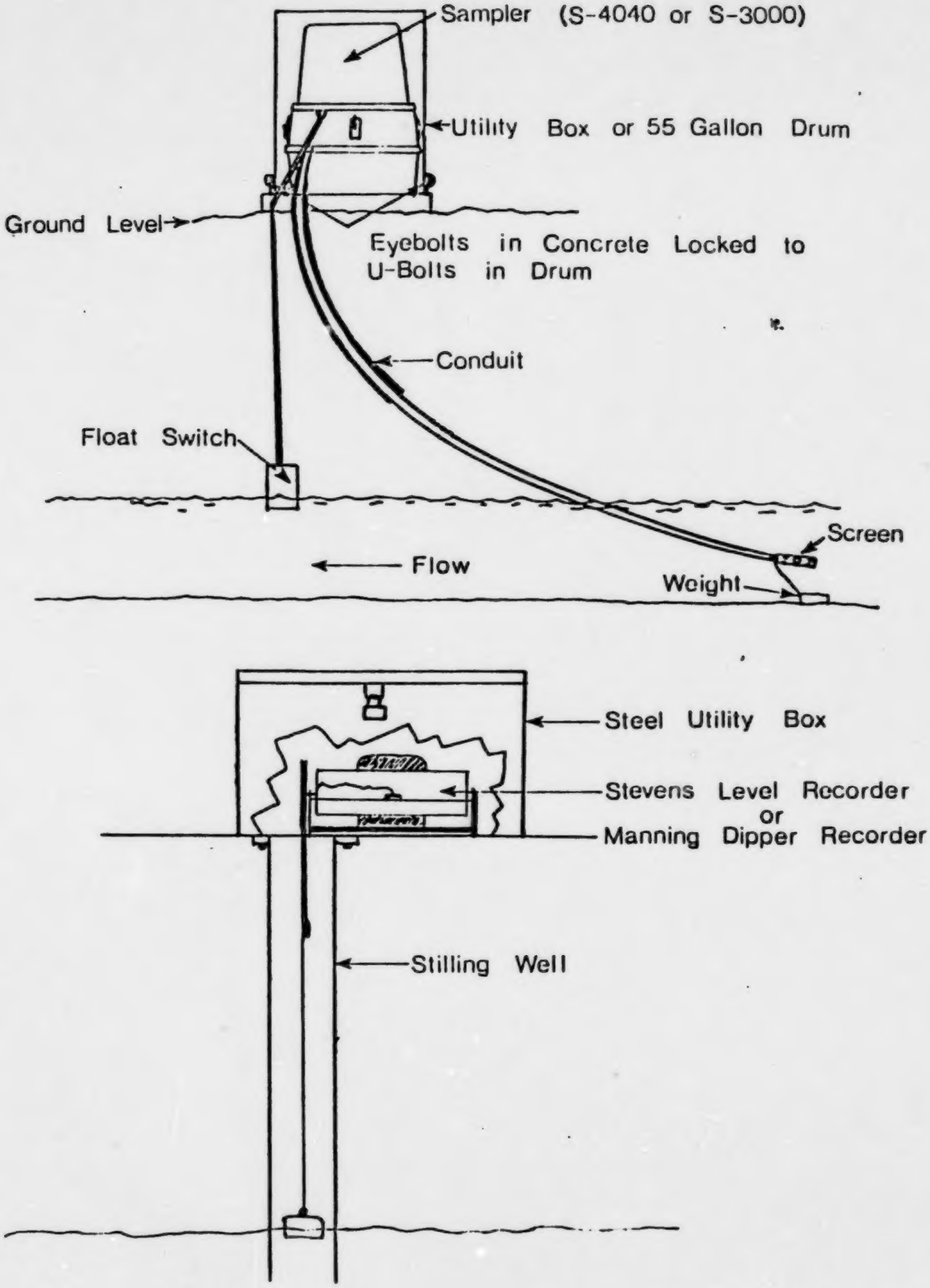
Sampler (S-4040 or S-3000)
Utility Box or 55 Gallon Drum
Ground Level
Eyebolts in Concrete Locked to U-Bolts in Drum
Conduit
Float Switch
Screen
Flow
Weight
Steel Utility Box
Stevens Level Recorder or Manning Dipper Recorder
Stilling Well

Figure IV - 6 Station Installation Schematic

Station 2 (Industrial Area)

Located at the end of the Genesee cut off and General Motors branches, this station was used to monitor industrial runoff from the southeast portion of the district (85 acres). Stormwater was sampled from a 54" pipe that receives stormwater and industrial drain overflow when the capacity of an upstream 10" force main (4 cfs) and storage capacity created by a 14" bulkhead in the 54" pipe downstream of the force main and upstream of Station 2 is surpassed. The manhole used for sampling is on General Motors property, approximately 3 feet from the perimeter fence. For security purposes and to gain access to equipment, a fence enclosure was built over the manhole with a gate on the perimeter fence. An ultrasonic flowmeter was housed in a metal box on the ground surface. The sampler itself rested on a shelf in the manhole.

Station 3 and 4 (Off-Line Detention BMP)

Station 3, located downstream from the off-line dry basin in the lower section of the Dryer Edgemont Branch, represented the outlet of this BMP. Station 4, representing the inlet to the BMP, receives drainage from 26 acres of mixed land use. In addition, the effluent from the in-line retention basin also is upstream of station 4. The drain at Station 3 drains an additional 16 acres of overland runoff which directly enters the basin.

Samples were collected from 36" pipes at both stations, using samplers were housed in utility boxes. The dipper unit at station 4 was placed on a shelf in the manhole, while the ultrasonic recorder at Station 3 was contained in the utility box. Standing water at both stations required that samples be collected based on a time basis and then flow composited manually. A float switch in the manholes at both stations activated the sampler when there was a rise in water level. The samplers were not driven by the flow recorders.

Station 5 and 6 (In-Line Retention Basin)

Station 6, located at the inlet of the golf course BMP, represents runoff contributions from an area of 32.5 acres of mixed land uses. Station 5, located downstream of the BMP, drains an additional 36.4 acres, representing drainage from the golf course into the BMP itself. Samplers were housed in utility boxes at both stations and were driven by the flow recorders. A dipper flow recorder at Station 5 was installed on a shelf in the manhole. An ultrasonic flow recorder was placed in the utility box at station 6. A Palmer-Bowles flume was installed at station 6 to provide a primary device for flow measurements.

Stations 7, 8, 9, and 10 (Upsized Pipes)

These stations were located on Grace Street and were used to monitor the inlet and outlet of the two BMP's on that street. From South to North, the direction of flow, Station 10 monitors the inlet and Station 9 monitors the outlet of the 144 foot BMP. Station 8 monitors the inlet and 7 monitors the outlet of the 85 foot BMP. The land use contributing runoff to the inlet of each BMP is the same as the corresponding outlets; however, each BMP receives contributions from different land uses. The drainage area above stations 9 and 10 is 74.9 acres and a total of 163.9 acres drains through stations 7 and 8 including the 749 acres from station 9.

All of the equipment at these stations was housed in manholes because all sampling locations were in the middle of the roadway. All of the samples were paced by flow meters at these stations. A Palmer-Bowles flume was installed in the 48" pipe at station 10 to provide a primary flow device for the BMP. A rectangular weir was placed at station 7 as the primary device for the downstream BMP.

Grand River Water Quality Stations

The Grand River was sampled once each month at three locations. Sampling sites were located at the Creyts Road Bridge, the Logan Street Bridge, and at the Webster Road

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Bridge. The Creyts Road site is upstream from Lansing and represents baseline conditions in the river as it enters the urban area. The Logan Street site is just upstream from the study area outlet, and the Webster Road Site is downstream from the outlet. The Michigan Department of Natural Resources (MDNR) takes samples from the Creyts and Webster sites, but not from the Logan Street station.

Grab samples were taken once each month on the same day that the MDNR obtained samples. The samples were taken on the downstream side of the bridge at the mid-point of the river, the sampling method corresponding to the method used by the MDNR. Sampling in the Grand River was initiated in November, 1979.

Rainfall Stations

The automatic rain gauge was located at Station 5 at the outlet to the in-line wet retention basin. This station was the primary station for monitoring rainfall volume and intensity for each event. The station operated continuously during the study except for occasional malfunctions. A backup recording rain gauge was located at the wastewater treatment plant immediately adjacent to the district boundary near station 1. Two manual rain gauges wee also maintained during the study. These gauges were located at the atmospheric deposition stations. These gauges were read and maintained as part of the atmospheric deposition program.

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Atmospheric Deposition Stations

Two atmospheric deposition monitoring stations were maintained during this project. Each station consisted of an automatic rainfall collector, a dust bucket, and a manual rain gauge. One station was located on the roof of the cemetary administration building adjacent to sampling station 1. The other station was located on the roof of the township hall on Michigan Avenue in the south central portion of the district. Installing these stations on the roof of a building was not considered ideal from a sampling viewpoint. However, the roofs were selected for security reasons since the equipment for these stations was very vulnerable to vandalism.

SAMPLING PROTOCOL

General

The field sampling program was designed to obtain data which would serve as the basis for the evaluations outlined in the objectives. The equipment and station locations described earlier in this chapter were utilized to collect the samples and data for the field program. The various aspects of this sampling program are described herein.

Flow Monitoring

Flow measurements were obtained at all ten stations in the drainage district (stations 1-10) for each event during which water quality samples were collected. Flow measurements were recorded on a continuous basis and totalized using the various flow recorders. Three basic methods were used to obtain the flow data.

A primary flow device was installed in each BMP to provide sound flow data. Palmer-Bowles flumes were installed at stations 6 (retention basin) and station 10 (south upsized pipe). A rectangular weir with end contractions was installed at station 7 (north upsized pipe). The Marsh McBirney unit which continuously measures depth and velocity was utilized at station 3 or 4 (detention basin) to provide flow data. The data from each of these flow measuring stations was considered the main flow measurement point for each BMP.

The Manning formula for open channel flow was utilized at stations 1, 2, 5, 8, 9 to relate depth measurements to flow. The formula provides a relationship for correlating depth with flow which can then be used with the flow recorder to both totalize volume and pace the sampling equipment. Applying this formula without field verification can result in poor results due to many site specific factors. Therefore, the formula was only used as the basis for developing the flow relationship. Actual field measurements were taken of the flow at varying depths by measuring both depth and velocity at these sampling stations. Both continuous recording of velocity and depth, and spot check measurements were then used to "calibrate" the formula. This data was then used to calibrate the field recorders to the site specific conditions. This method proved acceptable for all stations except stations three and four.

Both stations three and four had standing and some sediment water present at all times. The results of the velocity and depth measurements showed that the relationship of depth to flow did not follow the Manning formula relationship. Therefore a stage-discharge curve was developed for these stations. Level was recorded continuously and then the flow was calculated manually using the established flow-depth curve.

Runoff Water Sample Collection

Two types of sampling strategies were used at the ten runoff sampling stations-discrete sampling and composite sampling. Discrete sampling involved the collection of individual samples during the course of a rain event. Samples were actually flow proportional sequential composites, representing several time intervals during the event. The time intervals were adjusted to represent various portions of the hydrograph. Approximatelcy six samples were collected at each station for each event. Discrete sampling was generally performed 2-3 times at each station.

Flow weighted composite samples were collected for all other events. These samples provided data on mean concentration and total mass loads for each event. Samples were collected automatically by means of the samplers pulsed by the flow recorders. Samples were collected based on the volume throughput monitored at each station. This approach generates one sample per station for each event.

Grand River Sampling

The Grand River was sampled once each month at three stations. Grab samples were collected from bridges at each location. The sampling was performed to coincide with the State of Michigan Department of Natural Resources program. Two stations, Webster and Creyts Roads, were sampled by both groups. The middle station was only sampled by this project team. Initially, duplicate samples were collected and analyzed by both laboratories. The initial data indicated comparable results between laboratories after which time the sample at Creyts and Webster were only collected in order to provide analyses for parameters not monitored by the state program.

A check was performed on the single point grab sample method by sampling across the transect of the river at each station on a depth integrated basis. The results showed no significant differences between transect samples. Therefore, the mid river sampling points were used throughout the project.

Atmospheric Deposition Sampling

Samples were collected at the two monitoring stations to obtain information on the contribution of pollutants from dryfall and wetfall sources. Initially two types of samples were collected, namely bulk fallout (dry and wetfall) and wetfall during rain events. A third sample type, dry fallout, was added to the program during the last year of the study.

Bulk fallout samples were collected on a monthly basis in the dust bucket units. The dust buckets were open to the atmosphere from the first Tuesday of the month until the first Tuesday of the following month. Samples would then be retrieved and new buckets set in place. This timing was used to coincide with the National Atmospheric Deposition Program. Dust buckets were operated in a "wet" mode by maintaining water in the bottom of the bucket.

Wetfall samples were collected using the automatic samples which were open only during rain events. Two types of samples were retrieved for subsequent analyses-individual event samples and monthly composites. Individual rain event samples were simply removed from the collection unit following the rain event and transported to the laboratory for analysis. Monthly composite samples were obtained by combining the individual samples collected during each month into a single sample. Following each rain event, the individual sample would be removed from the unit. This sample would then be split into several bottles according to the analytical requirements and the appropriate preservative would be added to each sample bottle. Following each monthly period (first Tuesday to first Tuesday of next month) the composite samples would be delivered to the laboratory for analysis.

The atmospheric deposition collection program was the most problematic field program during this study. Several difficulties were encountered throughout the study. Problems were encountered with the sampling equipment, as well as with the results being obtained from the study. Because of the difficulties, several changes were made during the course of the project in an attempt to improve the sampling protocol. A discussion of the problems encountered, results of a review of sampling alternatives, and the various solutions attempted were reported in quarterly and annual reports submitted during the project.

ANALYTICAL PROGRAM

General

The analytical program followed during this study included the measurement of a wide range of potential pollutants which may be present in urban stormwater runoff. The program was designed to analyze various groups of constituents (solids, nutrients, metals, organics, oxygen demand, etc.) on a varying frequency for the various types of samples being collected. During the course of the project the frequency of some analytical measurements was adjusted either up or down depending on the findings of previous surveys. The analytical schedule did not change significantly during the period but some adjustments were made to reflect the results being obtained. The analytical list provided in this section reflects the basic program and frequency of analyses without attempting to give specific station by station information.

All of the chemical analyses were performed at our laboratories. The laboratory operates under a standard and strict Quality Assurance program as presented later in this section. All sample handling and analyses followed U.S. EPA approved methods where applicable.

Certain of the chemical and bacteriological analyses performed required initiation shortly after sample collection. All samples collected were transported directly to the laboratory as soon as possible (generally within 2-4 hours following collection). Analyses requiring immediate attention were analyzed upon receipt in the laboratory without regard to the time of day or night. Samples for other analyses were preserved according to approved procedures and analyzed within the normal laboratory schedule. Table IV-1 shows the sample groupings and preservative procedures.

Urban Runoff Samples (Stations 1-10)

The analytical program for the urban runoff samples was divided into analyses for flow composite samples and for discrete samples. The following chemical parameters were analyzed for 100% of the composite samples:

Total Solids (TS)
Total Suspended Solids (TSS)
pH
Alkalinity (alk)
Specific Conductance (SC)
Chloride

Sample I.D.	Analysis	Preservative/Container	Quantity
Group I	T.S. TSS pH + Total Alk. Spec. Conduct. Chloride Turbidity	4°C P,G.	1000ml
Group II	Particle Distribution	4°C P,G.	1000ml
Group III-S	Soluble TKN Soluble TOC Soluble T-P	Filter, then 0.3% H_2SO_4 4°C G	1000ml
Group III-T	TOC NH_3-N TKN NO_3 + NO_2-N G + O	0.3% H_2SO_4 4°C G	2000ml
Group IV	BOD	Freeze P,G	1000ml
Group V	COD	0.3% H_2SO_4 4°C P,G	50ml
Group VI	Total: - Lead, Iron, Zinc, Chromium, Copper, Nickel, Cadmium, Mercury, Arsenic	1% HNO_3 P	500ml
Group VII	PCB	25 ml hexane (Hexane-rinsed B+J bottle)	4 l.
Group VIII	Total Fluoride	4°C P,G	300 ml
Group IX	Orthophosphate	Filter, then Freeze P,G	50 ml
Group X	Phenolics	H_3PO_4 to pH< 2 and 1 scoop $CuSO_4$ 4°C G	50 ml
Group XI	Sulfide	2 ml Zn Acetate/500ml P,G	500 ml
Group XII	Pesticide scan	25 ml hexane hexane-rinsed B+J bottle	2-4 l.

Table IV-1 Sample Groups

Organic Carbon (Total, Soluble)
Phosphorus (Total, Soluble)
Kjeldahl Nitrogen (Total, Soluble)
Ammonia Nitrogen
Nitrate and Nitrite Nitrogen
Turbidity
Lead

The following parameters were analyzed in approximately 50% of the composite samples:

BOD_5
Chemical Oxygen Demand
Iron
Zinc
Chromium
Copper
Nickel
Mercury
Arsenic
Cadmium
Grease and Oil
Particle Size

The following parameters were analyzed in 10% of the composite samples:

Fluoride
Orthophosphate
Phenol
Total Coliform

Sulfide and Sulfate were analyzed in approximately 5% of the samples.

During discrete sampling events (approximately six samples per station) the following analyses were performed:

- TS
- TSS
- pH/alk
- Chloride
- Total Organic Carbon
- Total Phosphorus
- Total Kjeldahl Nitrogen
- Lead

During these events a flow composite sample was analyzed for the following:

- Ammonia
- Nitrite plus Nitrate
- Chemical Oxygen Demand

Other analyses included a screening evaluation of pesticide contamination of Station 1. All discrete samples, as well as periodic grab samples during composite events, were monitored for fecal coliform and fecal streptococcus. An organic priority pollutant analysis program also was performed at stations 2, 6, 10.

GRAND RIVER ANALYSIS

The Grand River analysis program was basically unchanged during the study. However. due to reductions in the MDNR budget. two additional analyses on the two State stations were added during the last year to maintain consistency within the data base generated during the initial portion of the program. The following parameters were analyzed for samples for each of the three river stations:

- Total Solids
- Total Suspended Solids
- pH
- Alkalinity
- Specific Conductance
- Chloride
- Turbidity
- Total Organic Carbon
- Total Phosphorus
- Total Kjeldahl Nitrogen
- Ammonia Nitrogen
- Nitrate and Nitrite Nitrogen
- Grease and Oil
- BOD
- Chemical Oxygen Demand

Fifty percent of the samples were analyzed for the following metals:

- Lead
- Iron
- Zinc

Copper
Chromium
Nickel

Cadmium, Mercury and Arsenic were also measured on selected samples.

Atmospheric Deposition Sample Analysis

All four sample types, individual rain event samples, monthly composite rain samples, bulk fallout and dry fallout were analyzed for the following constituents:

Total Solids
Total Suspended Solids
pH
Acidity
Specific Conductance
Chloride
Total Organic Carbon
Ammonia Nitrogen
Total Kjeldahl Nitrogen
Nitrate and Nitrite Nitrogen
Total Phosphorus
Chemical Oxygen Demand

The following metals were analyzed in 50% of the total number of samples:

Lead
Iron
Zinc

Chromium
Copper
Nickel

STATISTICAL ANALYSIS

As part of the evaluation procedures used in this study, a series of statistical methods were used in an attempt to describe and/or relate various storm characteristics and storm-generated pollutant loads. It was anticipated that if simple statistical methods could be used to properly relate the data, then powerful predictive tools would be available for aiding in the evaluation of area wide urban runoff problems.

Relevant storm characteristics and chemical parameters were selected for evaluation. The storm characteristics chosen. were rainfall volume. average rainfall intensity, maximum rainfall intensity, antecedent dry days, storm duration, runoff volume, and runoff coefficient. The chemical parameters selected were: total suspended solids. total phosphorus. total Kjeldahl nitrogen, biochemical oxygen demand, chemical oxygen demand, lead, copper, particle size distribution and removal efficiency. These characteristics and chemical constituents were chosen based on past studies and investigative results as the variables most likely to describe storm generated pollutant loads. In addition, the chemical constituents were chosen based on their being generally used in assessing water quality. An additional consideration was the fact that many of the other parameters measured during the study were very low in concentration and/or were below analytical detection limits. This led to the elimination of several parameters from more detailed evaluation. The above parameters were compiled for each monitored event at every station and entered into a computer file in the University of Michigan computer system. The data was then analyzed using the Michigan Interactive Data Analysis System (MIDAS) developed at the Statistical Research Laboratory of the University of Michigan.

The compiled data were first analyzed by obtaining a set of descriptive statistics: minimum, maximum, mean, standard deviation, median, geometric mean and geometric deviation. The distribution of the concentration data and other characteristics were then examined to determine if the data were normally distributed. Indications from NURP headquarters were that the pollutant data tended to follow log-normal distribution rather than normal Gaussian form. Evaluation of the tendency of the distribution included the generation of distribution plots and comparing mean and geometric mean values with the median.

The data was then tested using simple correlation analysis to test for the presence or absence of any significant linear relationships. The correlation coefficient produced, in this and all other bivariate analyses, is the standard Pearson product-moment correlation coefficient. Another, more complete, correlation analysis is available in MIDAS and was used frequently. This analysis produces the standard correlation coefficient plus confidence intervals -- the Student T-test and the attained significance. The T-test is the standard two-tailed, pooled variance test. The significance is a measure of the hypothesis that the correlation coefficient is equal to zero against the hypothesis that the correlation coefficient is not equal to zero. Therefore, if the significance of the correlation was larger than desired, the variables were considered to be unrelated.

For those relationships of interest, a least square linear regression was performed. MIDAS can analyze either multivariate or bivariate systems. In some cases it was of interest to test several related independent variables against a variable of interest; for instance, total suspended solids load against rainfall volume, intensity, and antecedent dry days. As a general rule such multivariate analyses produced a linear regression equation with a high degree of correlation. It was felt however, that the multivariate regressions were of an unwarranted complexity and not significantly more representative than the bivariate analyses presented in the text.

In the tables where linear regression statistics are presented, the reader will find the correlation coefficient, standard error, F-statistic and significance. Correlation coefficients are discussed above. The standard error is the error of the slope and is used in the determination of the hypothesis that the slope is equal to zero. The F-statistic is the standard F and is equal to the regression mean square divided by the error mean square. The error mean square is the square of the standard error. The F-statistic is used in testing the hypothesis that the slope of the regression line is not zero. For a simple bivariate linear regression. the F-statistic is equal to the square of T, where T is the usual two-tailed Student T-test.

As noted above, the standard error and F-statistic are used to test the hypothesis that the slope is equal to zero:

HO: $m=o$

against the hypothesis that the slope is not equal to zero:

H1: $m \neq o$

If HO is true, F has an F-distribution with 1 and n-2 degrees of freedom. HO is rejected if F is too large or if T is either too large or too small. Equivalently, HO is rejected if the attained significance level of either T or F is smaller than the desired level. Here. since the T- and F- tests are equivalent, these attained significance levels will be the same. The significance given in the text is the attained significance level for the F-test of the hypothesis that the slope is equal to zero (HO: $m=o$).[3]

Scatter plots of the various dependent-independent variable pairs were also generated for use in conjunction with the regression tests. The scatter plots are a graphical representation of the data plotted using standard cartesian coordinates. The scatter plots provide a visual representation of all of the data for a given station in a condensed and easily inspected format. These plots also provide a method for subjectively defining trends in the data for those cases where the correlation coefficient is weak. Further, outlying values, which can dramatically affect the results, can easily be identified by inspection of the plots.

The results of the various statistical tests are presented in tabular format throughout the results portion of this report. The data shown in these tables represents the final summarized information obtained after many testing procedures in various combinations had been attempted. It is felt that the descriptive information given is the most common method of summarizing the results but in no way is represented as an exhaustive list of possible statistical testing methods. Scatter plots for the main dependent, independent variable pairs are presented in the appendix. These plots are included to provide an overview of all of the individual event data points for the selected parameters in a convenient summary form. These persons desiring to further examine the data can obtain all of the individual event results from the Data Report or by retrieval from the USEPA STORET System.

QUALITY ASSURANCE/QUALITY CONTROL

A specific quality assurance plan was developed for this project. The plan was based on the general company program and applied specifically to this study. The quality assurance program included quality control procedures for both the field and laboratory aspects of the project. The QA program was prepared in written format at the beginning of the work and submitted as a special report to the sponsoring agencies. The full document has been filed with, and is available at, the Tri-County Regional Planning Commission, Ingham County Drain Commission, USEPA Region V, and NURP headquarters. Further detail (beyond the summary below) on the QA program is presented in the Appendix.

The field QA program included both qualitative and quantitative methods for controlling the field monitoring program. Qualitative aspects of the plan included areas of personnel training, equipment installation and maintenance procedures, and record keeping. An execution plan for the field work was prepared which detailed sampling and maintenance procedures in addition to call schedules, diagrams of equipment, and sampling crew structure. Quantitative aspects of the plan included collection of sample blanks, sample duplicates, grab samples for comparison, full composite duplicates, and preservative blanks. In addition, manual flow measurements using velocity and depth were used to periodically check flow recorders and frequent calibration checks were performed on all equipment.

Laboratory quality assurance procedures included both the control of measurements and control of laboratory operations. The control of analytical measurements included the use ot calibration standards, method blanks, sample duplicates, sample spikes, and blind check samples. Control of laboratory operations included control measures such as frequent maintenance and calibration of equipment, purity of chemicals and reagents, quality of glassware, uniformity of methods, and training programs. All of these measures were in place during the project in order to provide analytical results. Further results of the QA program are provided in the Appendix.

SECTION V

BMP EVALUATION

ON-LINE RETENTION BASIN

Introduction

The land use, basin geometry and monitoring station set up for the on-line retention basin are described in Section III of this report. The basin was designed to provide both downstream flood relief and water quality improvement. Both objectives were accomplished by restriction of outlet flows to a maximum of 2-3 cfs under pressure flow conditions. The restricted outflow increases detention time thereby facilitating the sedimentation process. As was stated in Section II, sedimentation was expected to be the primary method of water quality improvement.

Data Collection

The basin was monitored 29 times during the study period. Twenty six events were monitored using the flow weighted composite method, while three events were monitored in the discrete mode. Sampling was performed with either a Manning S-3000 composite sampler or a Manning S-4040 discrete sampler, each being paced by a flow meter. Composite samples were collected in a 15 liter polyethylene jug. Discrete samples were collected in one liter polyethylene bottles, with 24 bottles to a rack. In addition to the 29 events monitored at both inlet and outlet stations, the inlet was monitored during another 5 rain events and the outlet during one additional event.

All samples were handled and preserved using approved USEPA procedures. Samples were split and preserved in the laboratory within approved holding times or were preserved in the field. Analytical procedures followed approved methods. Quality control and quality assurance procedures are outlined in Chapter IV.

Results

Complete summaries of pollutant loads are presented in the Data Report. The summaries contain event mean concentrations, total mass, and mass per acre for each parameter monitored. In addition, rainfall and runoff summaries are presented for each station. The data represents only storm generated loads at the basin inlet (Station 6) as there is usually no base flow. The basin outlet (Station 5) had a small amount of base flow (usually <0.1 cfs) and this is included in the Event Summaries. Sampling was initiated when flow exceeded baseline conditions and ceased when it returned to zero or when a flow equal to 10 percent of the peak flow was reached. A brief statistical summary of rainfall, runoff, and selected parameter concentrations is presented in Tables V-1 and V-2.

TABLE V-1

Summary of Event Characteristics
Station 6 Basin Inlet

Parameter	n	Min.	Max.	Mean	Standard Deviation
Rainfall in	34	0.14	1.05	0.51	0.26
Runoff Volume Ft.3	34	3,000	52,000	20.674	11,262
Runoff Coeff.	34	0.21	0.65	0.38	0.099
TSS mg/l	34	6.0	495	84	97
T-P mg/l	34	0.011	0.76	0.20	0.15
TKN mg/l	32	0.53	4.8	1.5	0.84
BOD mg/l	20	2.0	23	8.8	5.4
COD mg/l	26	6.0	160	62	39
Lead mg/l	23	0.02	0.42	0.11	0.1
Copper mg/l	16	0.005	0.06	0.02	0.01

TABLE V-1 (Continued)

Summary of Event Characteristics
Station 6 Basin Inlet

Parameter	Median Conc.	Antilog 10 Mean	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	52	53	1.7250	0.43045
T-P	0.12	0.16	-0.80361	0.32649
TKN	1.3	1.4	0.13200	0.19494
BOD	7.1	7.5	0.87598	0.25415
COD	49	51	1.7100	0.30377
Lead	0.08	0.09	-1.0604	0.33222
Copper	0.01	0.01	-1.8879	0.24160

TABLE V-2

Summary of Event Characteristics
Station 5 Basin Outlet

Parameter	n	Min.	Max.	Mean	Standard Deviation
Rainfall in	29	0.14	1.02	0.50	0.24
Runoff Volume ft.3	29	3,000	68,000	26,414	15,824
Runoff Coeff.	29	0.04	0.35	0.22	0.077
TSS mg/l	28	2.0	44	11	8.4
T-P mg/l	29	0.007	0.1	0.06	0.02
TKN mg/l	29	0.01	1.7	0.93	0.37
BOD mg/l	18	0.48	8.9	3.8	2.2
COD mg/l	25	14	44	30	6.8
Lead mg/l	19	0.002	0.02	0.008	0.005
Copper mg/l	16	0.002	0.02	0.006	0.005

TABLE V-2 (Continued)

Summary of Event Characteristics
Station 5 Basin Outlet

Parameter	Median Conc.	Antilog 10 Mean	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	10	8.4	0.92672	0.31100
T-P	0.06	0.05	-1.2998	0.23239
TKN	0.94	0.74	-0.13117	0.44794
BOD	3.0	3.0	0.47679	0.34826
COD	32	30	1.4725	0.11076
Lead	0.008	0.006	-2.1950	0.31743
Copper	0.005	0.005	-2.2777	0.26827

Runoff Characteristics

The on-line retention basin was monitored over a wide range of rainfall amounts (0.14 - 1.05 inches). The range of observed runoff volumes displayed great latitude as well, with a minimum inlet and outlet volume of 3,000 cubic feet, and a maximum inlet volume of 52,000 cubic feet and a maximum outlet volume of 68,000 cubic feet. Typically inlet and outlet volumes were approximately equal, however on occasion the outlet volume exceeded the inlet volume by substantial amounts. For example, during an event on September 3, 1981 the inlet recorded 31,000 cubic feet while the outlet discharged 68,000 cubic feet.

As noted earlier, the outlet of the basin drains an extra 37 acres of parkland. During "typical" events little or no overland runoff is generated due to rapid infiltration. However, during early spring events and after prolonged periods of rain, the area surrounding the basin can become saturated and significant amounts of runoff and/or groundwater infiltration to the basin can be generated. In the event mentioned above, 2.52 inches of rain fell in the preceeding week, followed by one dry day, then another 1.05 inch rainfall occurred. It is assumed that the extra 37,000 cubic feet measured at the outlet originated via the above mentioned mechanism.

A linear regression of total runoff volume at the basin inlet against total rainfall was calculated (see Figure V-1 and Table V-5). The regression has a correlation coefficient of 0.91. In comparison with other stations in the study area, the on-line retention basin shows a high

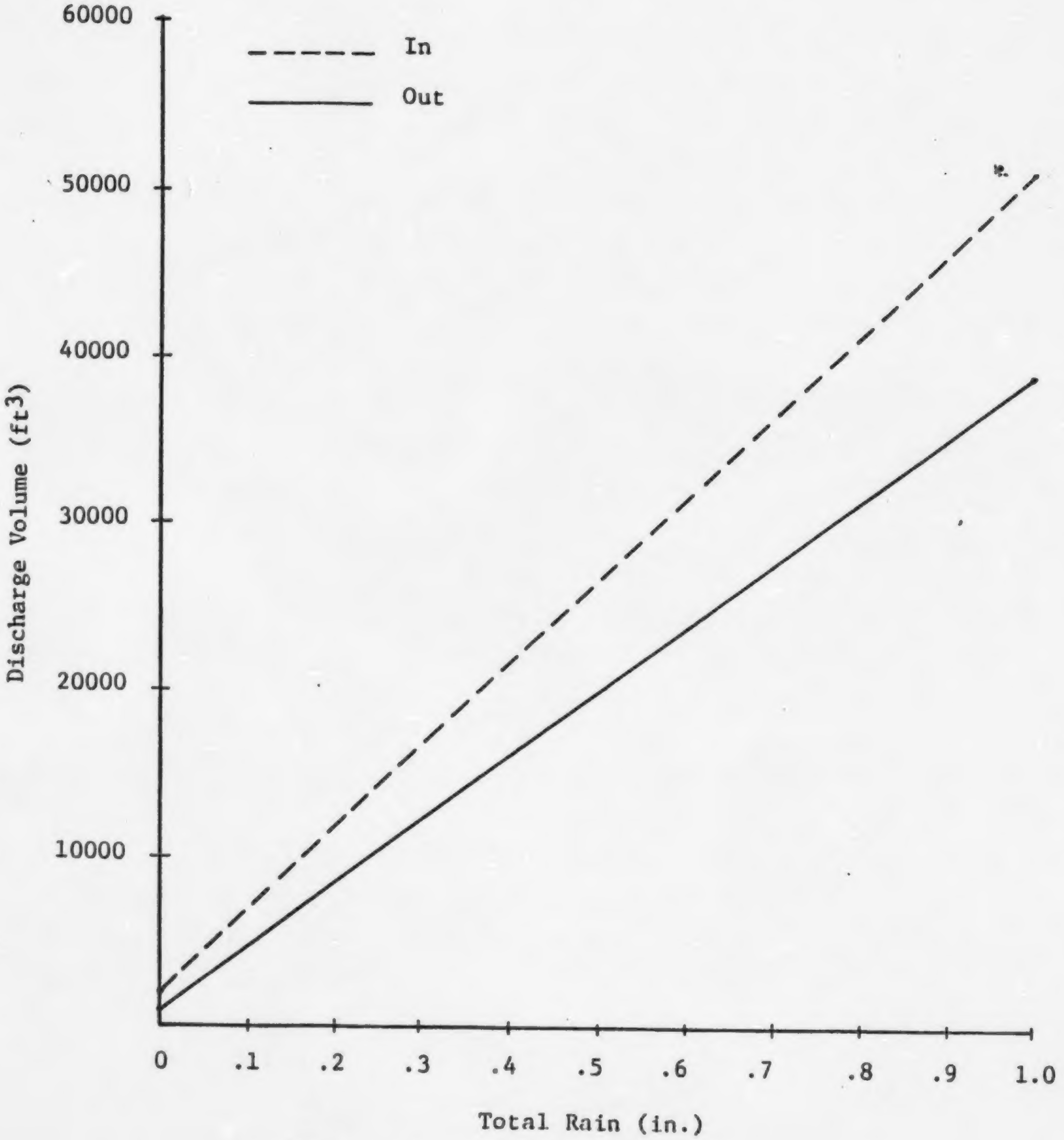

Figure V-1 Discharge Volume vs. Total Rain

degree of linear correlation of volume to rainfall. This high degree of correlation could be expected because a large percentage of the basin inlet drainage area is impervious. The inlet drains 30.3 acres of mixed residential and commercial land use, of which 68% is impervious.

The runoff coefficient ranged from 0.21 to 0.65, averaging 0.38 for all monitored events during the study period. A linear regression of the runoff coefficient against rainfall volume indicated no significant relationship (see Table V-3). Multivariate linear regressions were performed to see if the runoff coefficient is influenced by other weather conditions. Antecedent dry days, average and maximum rainfall intensity, and storm duration were included with total rainfall in the analyses. The multivariable regressions displayed no statistically significant relationships. The above analyses indicate that the runoff coefficient can best be represented by a study period average.

Removal Efficiency

During any given storm event as many as 30 chemical parameters were monitored. For the purposes of this analysis six representative parameters -- total suspended solids (TSS), total phosphorus (TP), total Kjeldahl nitrogen (TKN), biochemical oxygen demand (BOD), chemical oxygen demand (COD), lead (Pb), and copper (Cu) -- will be discussed in this section. The parameters will illustrate the type of removal that can be expected for solids, nutrients, and metals in a basin of this sort.

TABLE V-3

Linear Regression Statistics
Station 6 Basin Inlet

Dependent Variable	Independent Variable	n	Correlation Coefficient	Standard Error	F Statistic	Significance*
Runoff Coefficient	Total Rain	34	0.06	0.10	0.119	0.7323
TSS Concentration	Total Rain	34	0.12	98.21	0.456	0.5042
T-P Concentration	Total Rain	34	0.18	0.15	1.129	0.2959
TKN Concentration	Total Rain	32	0.11	0.85	0.350	0.5585
BOD Concentration	Total Rain	20	0.39	5.08	3.292	0.0863
COD Concentration	Total Rain	26	0.37	37.27	3.743	0.0649
Lead Concentration	Total Rain	23	0.01	0.10	0.002	0.9626
Copper Concentration	Total Rain	16	0.29	0.012	1.305	0.2724

* See Statistical Analysis for a discussion of significance.

The intent of this study was to monitor inlet and outlet concentration, and mass load of pollutants generated during storm events and determine the removal of pollutants in the basin. To this end, each parameter was evaluated using a series of statistical analyses. The results of the statistical analyses are given in Tables V-1 through V-6.

Water quality improvement in the retention basin occurs primarily through settling of suspended solids. Treatable nutrients and metals will generally be insoluble and/or associated with these solids. Therefore, it can be expected that overall water quality improvements will be related to the sedimentation process. The following segments of this report will discuss removal of selected parameters, their relation to rainfall characteristics, and the sedimentation process within the basin.

Total Suspended Solids

Removals of TSS ranged from net export to 99 percent removal over the course of the study. However, overall removal efficiency was usually very high, on the order of 80-90 percent. Mass loads and concentrations were highly variable at both the inlet and outlet of the basin. TSS inlet loadings ranged over two orders of magnitude, (from 4.3 to 460 pounds). Average inlet concentration (Event mean concentration) ranged from 6 to 495 mg/l. Basin effluent loads covered three orders of magnitude (0.75 to 160 pounds), with an event mean concentration range of 2-44 mg/l. Tables V-1 and V-2 contain a description of inlet and outlet parameters for all events.

TABLE V-4

Linear Regression Statistics
Station 5 Basin Outlet

Dependent Variable	Independent Variable	n	Correlation Coefficient	Standard Error	F Statistic	Significance*
TSS Concentration	Total Rain	28	0.31	8.11	2.703	0.1122
TP Concentration	Total Rain	29	0.07	0.02	0.117	0.7352
TKN Concentration	Total Rain	29	0.05	0.37	0.081	0.7775
BOD Concentration	Total Rain	18	0.02	2.29	0.010	0.9232
COD Concentration	Total Rain	25	0.36	6.46	3.527	0.0731
Lead Concentration	Total Rain	19	0.16	0.005	0.456	0.5086
Copper Concentration	Total Rain	16	0.08	0.005	0.081	0.7805
TP Removal	TSS Removal	28	0.68	26.51	22.64	0.0001
TKN Removal	TSS Removal	26	0.48	27.26	7.054	0.0138
BOD Removal	TSS Removal	18	0.53	29.98	6.232	0.0238
COD Removal	TSS Removal	23	0.76	20.18	28.211	0.0000
Lead Removal	TSS Removal	19	0.48	20.89	5.090	0.0375
Copper Removal	TSS Removal	14	0.68	27.59	10.433	0.0072

* See _Statistical Analysis_ for a discussion of significance.

TABLE V-5

Linear Regression Equations
Station 6 Basin Inlet

Equation	n	Correlation Coefficient	Standard Error	F Statistic	Significance	Equation Number
Discharge(ft^3) = 38468r + 908	34	0.91	4802	149.5	0.0000	(1)
ETSS(lbs.) = 128r + 3	34	0.91	15.94	149.5	0.0000	(2)
TSS(lbs.) = 130r + 38	34	0.32	104.02	3.6	0.065	(3)
ETP(lbs.) = 0.29r + 0.0068	34	0.91	0.036	149.5	0.0000	(4)
TP(lbs.) = 0.3r + 0.086	34	0.39	0.188	5.06	0.0214	(5)
ETKN(lbs.) = 3.19r + 0.075	34	0.91	0.398	149.5	0.0000	(6)
TKN(lbs.) = 3.32r + 0.176	32	0.75	0.762	39.1	0.0000	(7)
EPb (lbs.) = 0.21r + 0.005	34	0.91	0.026	149.5	0.0000	(8)
Pb(lbs.) = 0.26r + 0.017	23	0.48	0.138	6.39	0.0196	(9)

Note: r = Rainfall in inches.
E parameter = Estimated load based on average EMC and the discharge equation.

TABLE V-6

Linear Regression Equations
Station 5 Basin Outlet

Equation	n	Correlation Coefficient	Standard Error	F Statistic	Significance	Equation Number
Discharge(ft.^{3}) = 49121r + 1921	29	0.76	10840	36.11	0.0000	(10)
ETSS(lbs.) = 26r + 1	29	0.76	5.56	36.11	0.0000	(11)
TSS(lbs.) = 56.3r - 6	28	0.44	28.4	6.44	0.018	(12)
ETP(lbs.) = 0.18r + 0.007	29	0.76	0.040	36.11	0.0000	(13)
TP(lbs.) = 0.17r + 0.01	29	0.53	0.067	10.2	0.0036	(14)
ETKN(lbs.) = 2.3r + 0.09	29	0.76	0.487	36.11	0.0000	(15)
TKN(lbs.) = 3.1r - 0.08	29	0.69	0.815	24.4	0.0000	(16)
EPb(lbs.) = 0.02r + 0.008	29	0.76	0.004	36.11	0.0000	(17)
Pb(lbs.) = 0.03r + 0.001	19	0.46	0.014	4.59	0.0469	(18)

Note: r = Rainfall in inches.
E parameter = Estimated load based on average EMC and the discharge equation.

Linear regressions of TSS concentration and load against total rainfall were performed. The concentration of TSS was found to be independent of rainfall at both inlet and outlet. The linear regression of TSS load against rainfall indicated that the correlation was not strong (Inlet, r = 0.32; Outlet, r = 0.44), however, the significance of the regression indicates that while the line does not fit the data very well, there is a trend. A scatter plot of TSS versus rainfall shows a definite trend toward larger loads with increasing rainfall. However, as noted above, the large amount of scatter does not provide a good correlation (Table V-5, V-6). Figure V-2 presents a graphic representation of TSS load as a function of rainfall.

Total suspended solids concentration appears to be independent of rainfall. The data indicates that influent concentration is log normally distributed with geometric mean EMC equal to 53 mg/l (Table V-1). Outlet concentration is not well described by either log normal or Gaussian distribution; Table V-2 presents the arithmetic mean, the geometric mean and the median concentration.

The concentration mean EMC's were converted to load and a linear regression analysis performed against rainfall using the discharge equation. The results are presented in Tables V-5 and V-6 and shown graphically in Figure V-2. Figure V-2 presents actual and estimated TSS loads for a given rainfall.

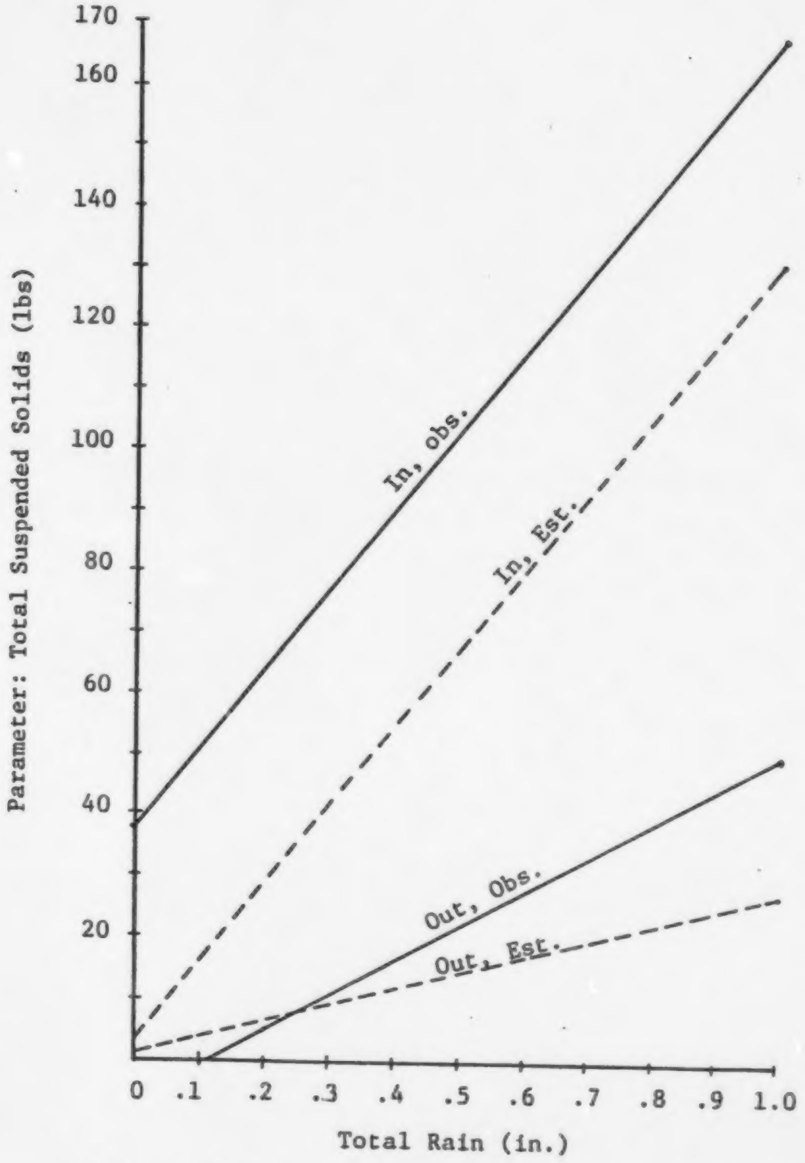

Figure V-2 Total Suspended Solids vs. Total Rain

Particle size distribution was examined to determine whether there existed a relationship between this parameter and removal efficiency. Regression and correlation analyses indicated no significant relationship between particle size distribution and removal efficiency. Nor was there any statistically significant relationship between particle size distribution and total rainfall. Particle size data shows a clear shift in distribution between the inlet and outlet of the basin, with the outlet containing a higher portion of smaller particles. Table V-7 presents study period mean particle size distribution as a percentage of the total concentration.

Total Phosphorus

Total phosphorus (TP) removal was highly variable, from net export to 94% removal. Inlet phosphorus load and concentration are independent of rainfall based on linear regression and correlation analyses. Inlet concentrations ranged from 0.011 to 0.76 mg/l and are log normal in distribution, the geometric mean being 0.12 mg/l. Total phosphorus load and concentration are independent of rainfall at the outlet as well, with geometric mean concentration being 0.05 mg/l (see Tables V-1 and V-2).

Total phosphorus at the inlet is primarily in the insoluble form, as shown by a correlation of 0.87 at the 99% confidence limit between TP concentration and TSS concentration. At the outlet a far greater proportion of

TABLE V-7

Particle Size Distribution

Size	Inlet (%)	Outlet (%)
>125	5	1
125-62.5	9	3
62.5-10	49	32
10-4	20	16
4-1	11	32
<1	6	16

the total phosphorus is in the soluble form. The change in composition between the inlet and outlet would suggest that phosphorus removals should be commensurate with TSS removal. In fact, a scatter plot of TP removal versus TSS removal indicates that increasing TP removal accompanies increasing TSS removal. A linear regression of TP removal against TSS removal results in a correlation coefficient of 0.68, with a 99% confidence limit (see Table V-4). The linear regression and the scatter plot both indicate that TSS removal and TP removal are linked, however, there is considerable scatter in the data.

A mass balance of inlet TP and outlet TP over the course of the study period showed a removal of 50%. As noted above, mean concentration at the inlet is 0.12 mg/l and at the outlet 0.06 mg/l, which also indicates a removal of 50%. Figure V-3 presents inlet and outlet loads for TP, both observed and estimated. The estimated curve comes from geometric mean EMC and the discharge equations. See Table V-5 and V-6 for the linear regression equations and related coefficients.

Total Kjeldahl Nitrogen

Total Kjeldahl nitrogen (TKN) removal efficiency displays the same type of variability as TP. TKN was exported from the basin during 10 of the 29 monitored events, while, during one event 98 percent of the influent mass was removed. Inlet concentration of TKN is independent

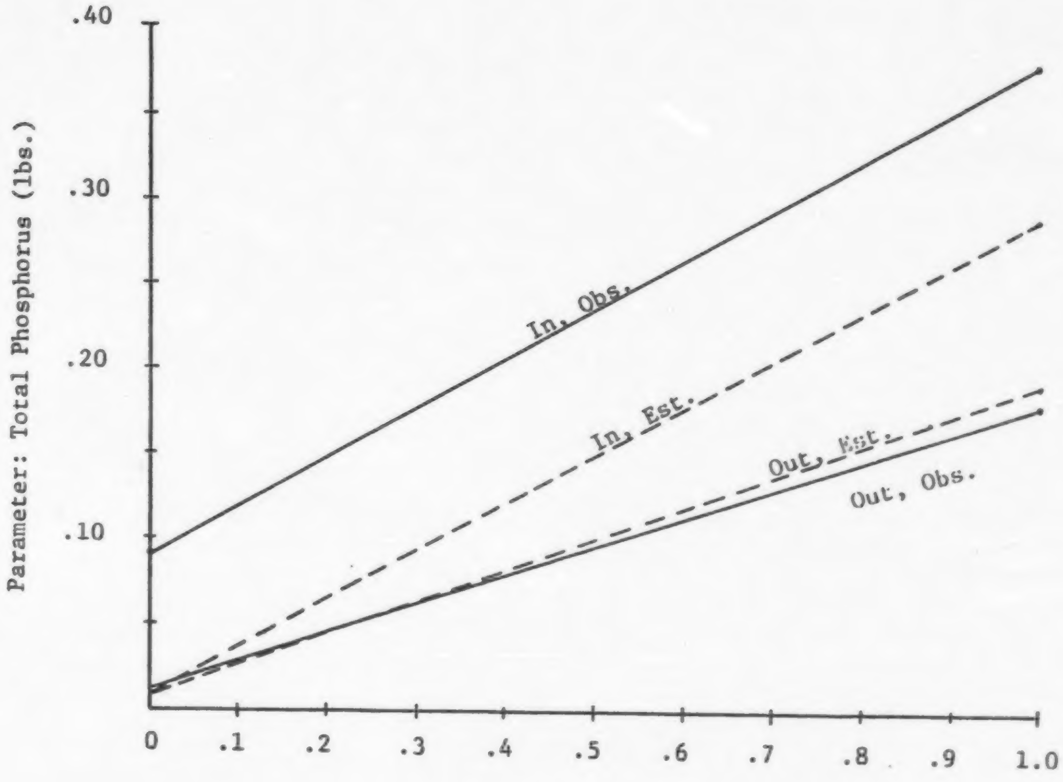

Figure V-3 Total Phosphorus vs. Total Rain

of rainfall and appears log normal in distribution. A linear regression of inlet load versus rainfall exhibited a correlation of 0.75 at the 99% confidence limit; a fairly high correlation relative to the other parameters discussed in this report. Outlet concentration was neither clearly log normal nor Gaussian in distribution. Concentration at the outlet fell in a narrow band and could be described equally well by either type of distribution. A regression of outlet load against rainfall showed a relatively high correlation (r=0.69) at the 99% confidence limit. Scatter plots of load and concentration versus rainfall at both stations confirm the trend. Tables V-1 and V-2 present concentration data, Tables V-3 through V-6 present linear regression output.

A mass balance of TKN input and output over the study period indicates 18% of the influent mass was removed by the basin. Using median concentration values, removal efficiency was observed to be 28%. Figure V-4 presents influent and effluent mass load curves for TKN. This figure includes actual loads based on a regression of observed values and estimated loads based on mean EMC values and the discharge equations. The estimated effluent line was calculated using the geometric mean EMC (0.74 mg/l).

Total Kjeldahl nitrogen concentration at the basin inlet displayed correlation of 0.86 with TSS concentration at the 99% confidence limit, while at the outlet the correlation coefficient was 0.29. This association of TKN with the particulate matter is confirmed by the percentage of soluble Kjeldahl nitrogen at the two stations, with the inlet exhibiting a much higher percentage of insoluble Kjeldahl nitrogen than the outlet.

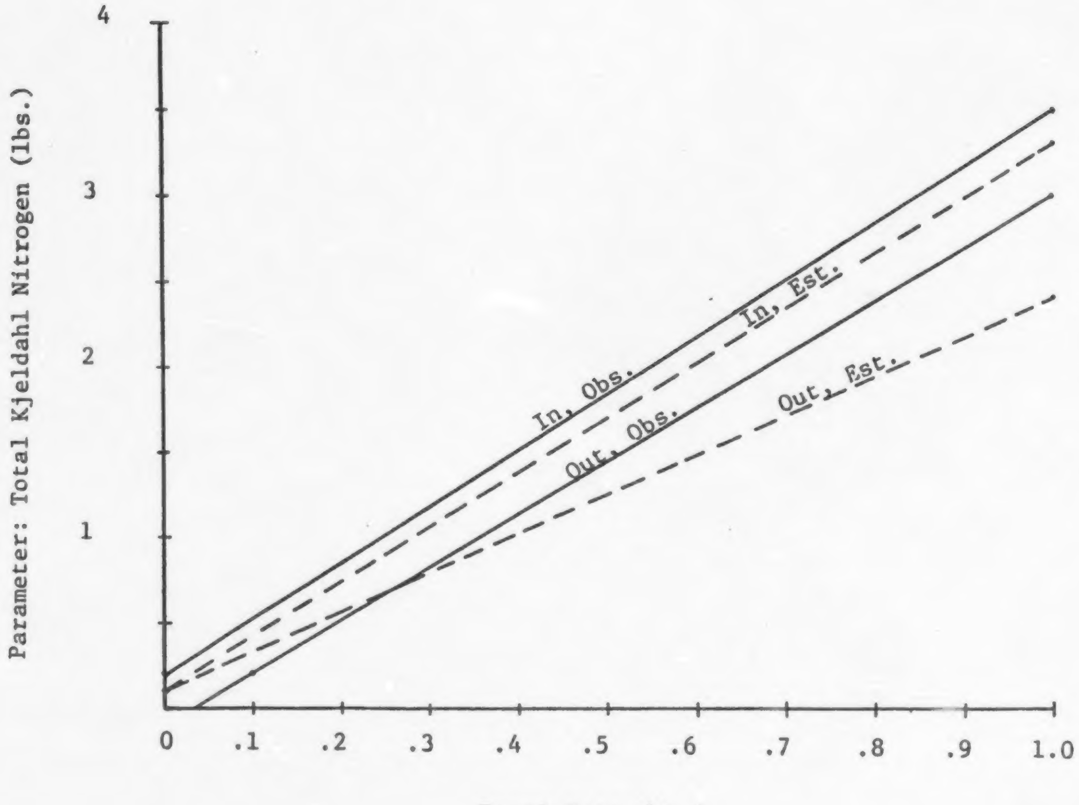

Figure V-4 Total Kjeldahl Nitrogen vs. Total Rain

Biochemical Oxygen Demand

Biochemical oxygen demand (BOD) was removed by the retention basin during 13 of 18 events. The largest observed removal of BOD was 87% during one event, while removals were greater than 70% for 7 of the 13 events. Based on a study period mass balance, 44% of the total influent mass was removed by the basin. Regression analysis of concentration and load indicated no statistically significant relationships to rainfall at either station (see Tables V-3 and V-4).

Influent and effluent concentrations can be best described by a log normal distribution. The geometric mean concentration at the inlet is 7 mg/l with a minimum of 2 mg/l and a maximum of 23 mg/l. The basin outlet had a concentration range of 1 to 16 mg/l and a geometric mean concentration of 3 mg/l. For a description of BOD concentrations see Tables V-1 and V-2. Tables V-3 and V-4 present the results of the linear regression statistics.

There appears to be a relationship between BOD and TSS at the basin inlet (r=0.71), but no corresponding relationship at the outlet (r=0.08).

Chemical Oxygen Demand

Chemical Oxygen Demand (COD) removal varied widely during the study period. Net export occurred during 5 of 23 events. During events when COD was removed, the removal efficiency ranged from 10 to 85%. As with the parameters mentioned previously, there is no statistically significant relationship of load or concentration to rainfall at either station (see Tables V-3 and and V-4).

Concentration at the basin inlet displays a log normal distribution, with geometric mean concentration equal to 49 mg/l and a range of 6 to 160 mg/l. Outlet concentration is not accurately described by either a log normal or Gaussian distribution. In both distributions the average concentration is equal to 30 mg/l and the median value is 32 mg/l. Concentration ranges from 14 to 44 mg/l at the outlet (see Tables V-1 and V-2).

A mass balance performed over the length of the study period shows a removal of 30% of the total mass input. Using geometric mean concentration values, removal efficiency was observed to be 38%.

Lead

Significant amounts of lead were consistently removed by the retention basin. More than 90% of influent lead was removed during 9 of 19 events, and was never exported. As is true with the other parameters in this study, concentration appears to be independent of rainfall at both the inlet and outlet. Inlet concentrations displays a log normal distribution with the geometric mean concentrtion equal to 0.09 mg/l. The concentration distribution at the outlet is not adequately described by either log normal or Gaussian function, although a log normal distribution is assumed. The geometric mean concentration at the outlet is equal to 0.006 (Tables V-1, V-2).

Based on a mass balance for all monitored events, the retention basin removed 90% of the influent lead load. Using the geometric mean concentration for inlet and outlext the removal efficiency was 93%.

Lead in stormwater is generally adsorbed on particulate matter. Therefore, it could be expected that lead removal would correlate well with suspended solids removal. A linear regression of lead removal versus TSS removal indicated that the two are not related at a high degree of correlation (r=0.48). On a percentage basis, lead removal is consistently higher than TSS removal (Table V-6). While there is weak correlation between TSS removal and lead removal, there is a correlation between concentration of suspended solids and lead concentration (r=0.91) at the basin inlet. However there is no similar correlation at the outlet (r=0.15). This would suggest that influent lead is mostly insoluble and adsorbed on particulates and is being flaculated and settled in the basin. In the basin water column there is a low level of background lead, measured at 0.0003 mg/l during one dry weather survey. Background lead plus that adsorbed on fine particulates is what is discharged from the basin.

As shown in Figure V-5 lead load at the outlet remains fairly consistent regardless of influent load. The equations used to plot the curves in Figure V-5 are presented in Tables V-5 and V-6.

Copper

Copper removals are more variable than those shown by lead, ranging from net export to 94% removal. The basin removed copper during 12 of 14 events. Copper concentration and load are independent of rainfall at both stations (Tables V-3 through V-6). Concentration at both inlet and outlet are best described by a log normal distribution with mean EMC at the inlet equal to 0.01 mg/l and at the outlet, 0.005 mg/l (Tables V-1, V-2).

A study period mass balance indicates 36% of influent mass was removed by the basin. Removal efficiency based on the geometric mean concentration was 50%.

Snowfall

The on-line retention basin was monitored during the melting of accumulated snow in 1981. During the monitoring period 160,000 cubic feet of runoff entered the basin and 370,000 cubic feet was discharged. The extra 210,000 cubic feet of runoff appears to have originated from the 37 acres

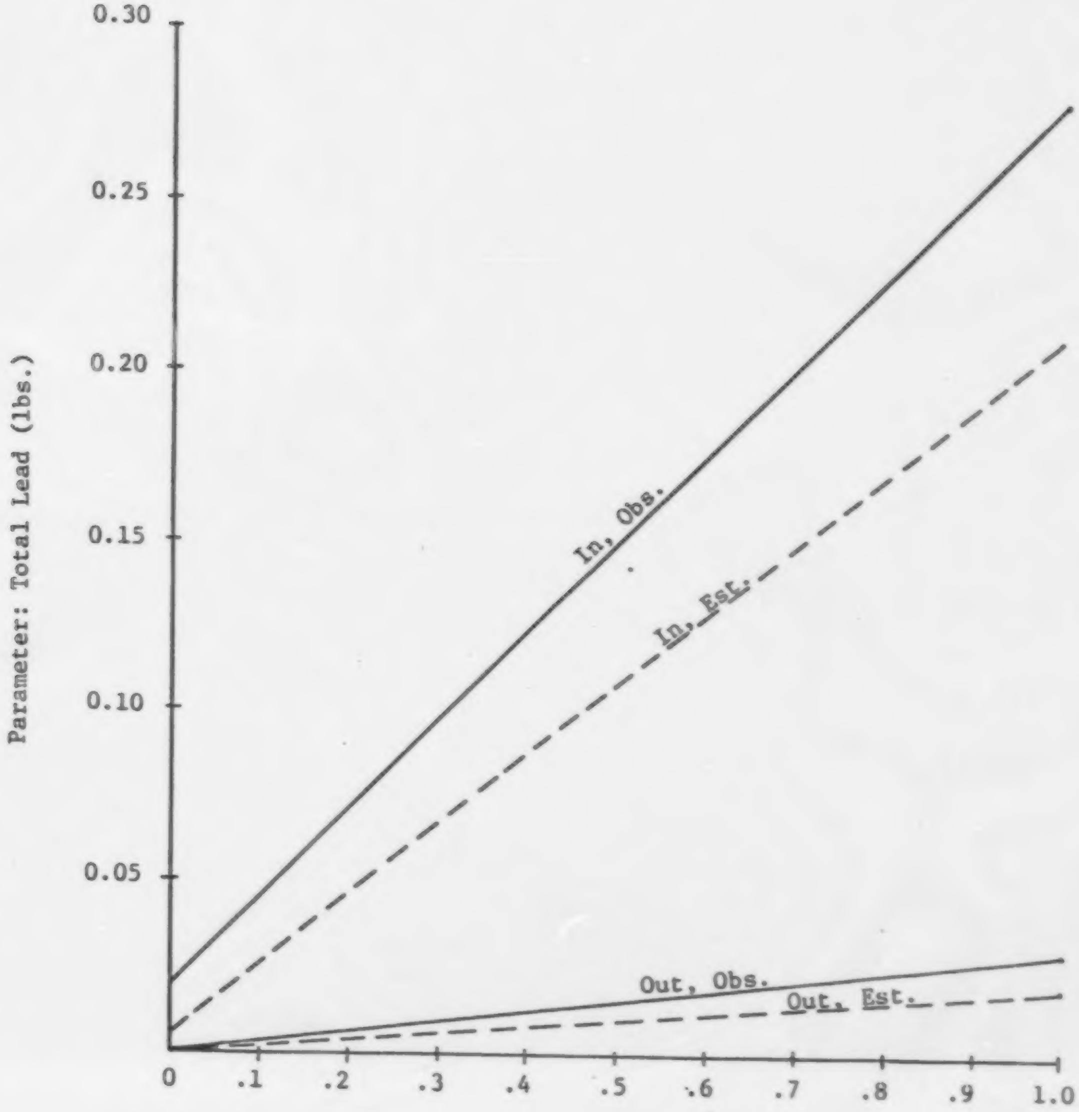

Figure V-5 Total Lead vs. Total Rain

of parkland surrounding the basin. The snowmelt was monitored over the course of 4 days. Runoff flow peaked between 1200 and 1400 hours each day and inlet flow usually lasted 6-8 hours each day. The outlet discharged continuously. As would be expected, the snowmelt did not produce the velocity or flow rates typically seen during a rainstorm. The inlet peaked at 20-30% of maximum capacity versus 70-100% often seen during rainstorms. The outlet, however, given the controlled outlet structure, did reach 100 percent of capacity, although the rising limb of the hydrograph has a much lower slope than that generally seen during a rainstorm.

The concentrations for all parameters, with the exception of influent TDS and chloride, fell well within the range observed during rainfall events. Infuent TDS and chloride were both an order of magnitude greater than that observed during rain events. Approximately half of the TDS can be accounted for by the chloride contributed by road salt.

The basin effluent displayed lower concentrations of refractory materials than would be expected given the influent concentrations. Influent chloride, for instance, was 980 mg/l while outlet concentration was only 220 mg/l. The most plausible explanation for the difference would be a dilution effect by snow melting around the basin proper and the water initially present in the basin (230,000 cubic feet). As noted above, an extra 210,000 cubic feet of runoff originated from the land surrounding the basin. This extra flow represents about 40% of the total flow and could be expected to be very clean as this area contains no roadway surfaces.

During the events that followed the snowmelt, influent concentration of TDS and chloride was fairly high, although lower than during the snowmelt. Levels declined through the rest of the summer as the final traces of road salt were washed off the highways. At the outlet however, TDS and chloride levels showed a marked increase in concentration during events that immediately followed the snowmelt. For instance, during the next event monitored after the snowmelt the chloride concentration at the outlet was three times that observed during the snowmelt. TDS values then began to decrease over the course of the summer. Obviously the snowmelt effluent was diluted by overland flow and as the snowmelt continued beyond the monitoring period. small amounts of very concentrated runoff contributed to the buildup of dissolved materials in the basin. This buildup and subsequent flushing of dissolved materials from the on-line retention basin is the only seasonal trend noted during the course of this study.

On a mass balance basis the basin removed 66% of influent TSS load and lowered the concentration from 110 mg/l to 17 mq/l. Eighty percent of the influent lead load was removed while lead concentration declined from 0.23 mg/l to 0.02 mg/l. The difference between load removal and removal based on concentration is due to the flow imbalance at the two stations. Similar concentration reductions were noted for other parameters, especially, the metals. Nutrients showed little or no reductions in most cases, however it is of interest that nutrients at the inlet usually had a greater fraction in the insoluble form than at the outlet. Nutrients at the outlet were almost completely in the soluble form (e.g., TP-100%, TKN-91%, TOC-100%).

Runoff during snowmelt periods contribute large quantities of pollutants and water to the on-line retention basin. The data indicates that the basin has the ability to withstand the stress and achieve a reduction of pollutants.

Discussion

It was anticipated that removal of pollutant materials in the basin would be accomplished primarily by sedimentation of suspended solids. The principal factor affecting sedimentation under ideal conditions is the detention time. Under such conditions detention time will determine the critical settling velocity and all particles having a settling velocity greater than the critical velocity will be removed. Actual performance of the basin will depend on the magnitude of deviation from ideal conditions due to short circuiting, turbulence and scour.

As noted in Section III, the basin has a restricted outflow structure that limits effluent flow to 2-3 cfs under pressure conditions. The structure attenuates the flow and increases detention time to allow sedimentation of storm generated solids. The restricted outflow also decreases the potential for short circuiting, scour and turbulence during peak flow periods.

Table V-8 presents minimum and average theoretical detention times for a range of storm sizes. Detention time

180

TABLE V-8

Average and Minimum Detention Time

Event Date	Rain Volume	Average Q (cfs)	Peak Q (cfs)	Ave. td. (hrs.)	Min. td. (hrs.)	Inf. TSS EMC	Eff. TSS EMC	TSS Removal Efficiency (%)
6/1/80	0.23	0.28	1.3	231	50	19	9	53
10/14/80	0.44	0.40	0.92	161	70	74	17	77
6/7/81	0.57	0.80	2.1	81	41	167	10	94
6/5-6/80	0.63	0.39	0.88	166	74	52	14	74
7/16/80	0.96	1.1	2.1	58	41	96	11	90

was calculated assuming plug flow. The data indicates that detention time, while important, is not the only consideration in determining removal of suspended solids. As shown in Table V-8 events with the longest detention time do not necessarily result in the greatest removal percentage. Inlet concentration and particle size distribution play an important role in the removal percentage obtained.

Two dry weather surveys were performed during the study. Background TSS levels were found to be 7 mg/l during one and 15 mg/l during the other. (The complete data set for the dry weather surveys can be found in the Data Report). The dry weather data suggests that there is a particle size fraction that does not settle regardless of detention time and also that there is a certain background concentration in the basin.

In the case of storms with a low influent TSS concentraton or storms which have a small percentage of settleable particle, the background concentration can become a significant portion of the effluent load and directly affect the percent removal calculation.

It would appear from the data collected over the two period that the effluent concentration from the basin is not dependent on the storm size, detention time, inlet concentration or other physical factors which normally would affect the sedimentation processes. The discrete events sampled showed that the concentration at peak flow periods was within the range of all of the events monitored. Thus, even high peak loading rates appear to have little affect on the effluent concentration.

Given the independence of the effluent concentration on all factors tested, the basin is apparently achieving the maximum level of control which can be attained within reasonable basin size by the sedimentation process. Indeed, the basin may actually be over-sized for the drainage area supplying runoff to the basin. The data which was collected does not allow for an evaluation of the impact that a smaller basin with higher outlet flows would have on removal levels. It is possible that the basin is designed at the edge of the size-removal relationship or that a significantly smaller basin could be utilized to achieve similar removals. Hopefully, as the data base for retention basin performance is increased during the NURP project, it will be possible to estimate an optimal basin size for given removal needs.

The steady nature of the effluent concentrations (i.e. narrow range of effluent concentration values) provides the basins for comparing discharge levels from the controlled non-point source with effluent levels typically required of wastewater treatment plants. Typical tertiary treatment standards set in the State of Michigan are shown in Table V-9 along with the retention basin mean values.

As can be seen the effluent from the basin is generally as good as or better than tertiary treatment standards. While this comparison does not indicate whether or not this effluent water quality will have a negative impact on a receiving stream, it does show that the basin effluent is comparable with the strict discharge levels currently being set for point source discharges.

TABLE V-9

Comparison of Retention Basin Effluent with Tertiary Treatment Standards

	Retention Basin	Wastewater Std.	
		max.	ave.
TSS mg/l	11	10	7
T-P mg/l	0.06	<1	<1
BOD_5 mg/l	4	10	7
NH_3-N mg/l	0.1-0.2	1-5	-
T-Pb mg/l	.008	-	-

Costs

Cost data for the On-line Retention Basin is presented in Table V-10. The costs of the inlet and outlet pipes include excavation, placing the pipe, backfilling with the appropriate material and compacting, fertilizing and reseeding the damaged area, and hauling the excess excavated material to a dump site one mile away. These costs were based on cost factors found in the 1980 Dodge Guide.[4] The costs for the manholes were taken from the contractors bid documents. Engineering design and inspection fees were assumed to be 11.2% of the construction cost, legal and financial fees were assumed to be 1% of the construction cost, and the contingency costs were assumed to be 6.8% of the construction cost. The costs of design review and approval were based on estimates of the time requirements of drain commission personnel. Since this BMP is located in a city-owned golf course, there were no right-of-way or site acquisition costs.

Routine maintenance costs assume that the BMP will be inspected ten times per year by drain commission personnel, at a cost of $60.00 per inspection. Waste disposal costs were determined by assuming that the retention basin will have to be dredged once every twenty-five years, and that 600 cubic yards of material will be removed each time. The cost of the dredging assumes that the basin will be excavated, loaded on trucks and hauled to a disposal site ten miles from the site. Cost factors for these activities

TABLE V-10

Cost Data for the On-line Retention Basin

Capital Cost (in 1980 Dollars)	
Inlet Pipe and Manholes	$23,500.00
Outlet Pipe and Manholes	19,300.00
Retention Basin	62,400.00
	$105,200.00
Engineering Design and Inspection	$11,800.00
Review and Approval of Design Plans	1,200.00
Legal and Financial Fees	1,050.00
Contingency	7,150.00
	$21,200.00
Total Installed Capital Cost	$126,400.00

Annual Operating Cost (in 1980 Dollars)	
Routine Maintenance	$600.00
Waste Disposal	350.00
Capital Cost*	6,320.00
Interest**	7,510.00
Total Annual Operating Cost	$14,780.00

* Straight-line depreciation of total installed capital cost over 20 years.

** Bonds at $5.9419%, principal of $126,400.00.

were obtained from the 1980 Dodge Guide. A dumping fee of $4.00 per cubic yard was used, this assumes that the dredged material will not require handling as a hazardous waste. The total cost of dredging and disposal of the material, in 1980 dollars, was $6,850. If an 8% rate of inflation is assumed, then in twenty-five years an amount of $46,950 will be required for this purpose. Assuming an interest rate of 12%, and a twenty five year amortization period, the annual cost of waste disposal is $350.

The annual capital cost was determined from straight-line depreciation of the total installed capital cost over a twenty year period. Annual interest costs assumed an interest rate of 5.9419% (the actual interest rate of the bonds sold to finance the Bogus Swamp Drain project) and a principal equal to the total installed capital cost.

IN-LINE UPSIZED PIPES

Introduction

The land use, station descriptions and design theory of the in-line upsize pipes were presented in Sections III and IV. As described earlier, the upsize pipes are in-series under the Grace Street portion of the district. The upsize pipes are designed to serve as sediment traps by slowing the water velocity, providing detention time, and allowing the sedimentation process to occur. The pipes provide a sump for deposition of settleable materials, which can then be removed by periodic cleaning.

Data Collection

Eighteen events were monitored at both inlet and outlet of the upstream unit. Discrete surveys were performed twice, while the remaining events were sampled using the flow proportional composite method. Because of equipment failure at one or the other of the stations, the inlet was monitored during two extra events and the outlet during four additional events.

Seventeen events were monitored concurrently at the inlet and outlet stations at the downstream upsize pipe. Two of the events were discrete surveys, while during the remaining events flow proportional composite samples were collected. A total of 23 events were monitored at the inlet and 21 at the outlet.

Samples were split and preserved in the laboratory, or, if the recommended holding time would be exceeded, preserved in the field in accordance with approved USEPA procedures. Analytical procedures followed approved methods. Quality control and quality assurance procedures are prsented in Section IV.

Results

Summaries of pollutant concentration, load, and load per acre are presented for each event in the separate Data Report. In addition a rainfall runoff summary for the event is given for each station.

A wide range of rainfall amounts and flows were monitored during the study period. The smallest rain monitored at any of the upsize pipe stations was 0.14 inches, while the largest was 1.63 inches. Runoff volumes varied from 3,000 to 135,000 cubic feet.

As noted previously, the two upsize pipes drain an area of mixed land uses. The upstream pipe drains 75 acres. with a 31 percent effective imperviousness. The downstream pipe drains another 89 acres (13% effective impervious), for a total of 164 acres of which 21% is effectively impervious. Runoff, as a percentage of rainfall, is approximately the same for both units. The mean runoff coefficient for both upsize pipes is 0.12.

Removal Efficiency

During any given storm event as many as 30 chemical parameters were monitored. For the purposes of this analysis seven representative parameters - total suspended solids (TSS), total phosphorus (TP), total Kjeldahl nitrogen (TKN), biochemical oxygen demand (BOD), chemical oxygen demand (COD), lead (Pb) and copper (Cu) - will be discussed in this section. The parameters will illustrate the type of removal that can be expected for solids, nutrients, and metals in a system of this sort.

The intent of this study was to monitor inlet and outlet concentration and mass load of pollutants generated during storm events, and to determine the removal of pollutants in the system. To this end, each parameter was evaluated using a series of statistical analyses. The results of the statistical analyses are given in Table V-11 through V-18.

It was anticipated that water quality improvement in the upsize pipes would occur primarily through settling of suspended solids. Treatable nutrients and metals will generally be insoluble and/or associated with settleable solids. Therefore, it can be expected that overall water quality improvements will be related to the sedimentation process. The following segments of this report will discuss removal of selected parameters, their relation to rainfall characteristics, and the sedimentation process with the upsize pipe system.

Total Suspended Solids

The upstream upsize pipe displayed highly variable removal efficiences during monitored events. Affect on suspended solids ranged from net export of material during three events to a high of 66% removal during the 14 events when removal occurred. Typically, suspended solids removals were in the 30-40 percent range. A mass balance calculated over the study period for all monitored events showed the upstream pipe removed 12% of the measured influent mass of 4,000 pounds of suspended solids.

TABLE V-11

Summary of Event Characteristics
Station 10 Upstream Pipe Inlet

Parameter	n	Min.	Max.	Mean	Standard Deviation
Rainfall (inches)	20	0.17	1.63	0.59	0.344
Runoff Volume (ft.^{3})	20	3,000	55.000	19,300	13,639
Runoff Coefficient	20	0.04	0.23	0.12	0.045
TSS mg/l	20	34	667	184	151.15
TP mg/l	17	0.11	1.42	0.45	0.362
TKN mg/l	16	0.38	4.10	1.67	0.908
BOD mg/l	9	3	8	5	1.688
COD mg/l	11	9	120	56	32.92
Lead mg/l	13	0.038	0.300	0.114	0.086
Copper mg/l	7	0.010	0.400	0.076	0.144

TABLE V-11 (Continued)

Summary of Event Characteristics
Station 10 Upstream Pipe Inlet

Parameter	Median Conc.	Antilog 10 Mean	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	120	137	2.1368	0.3466
TP	0.32	0.36	-0.4475	0.2819
TKN	1.40	1.46	0.16328	0.24457
BOD	5	5	0.71114	0.14117
COD	53	47	1.6727	0.30315
Lead	0.082	0.092	-1.0353	0.28761
Copper	0.023	0.029	-1.5312	0.54758

TABLE V-12

Summary of Event Characteristics
Station 9 Upstream Pipe Outlet

Parameter		n	Min.	Max.	Mean	Standard Deviation
Rainfall (inches)		22	0.14	1.63	0.50	0.309
Runoff Volume (ft.^{3})		22	3,000	55,000	16,591	0.115
Runoff Coefficient		22	0.04	0.29	0.13	0.056
TSS	mg/l	22	14	507	141	120.3
TP	mg/l	19	0.16	1.51	0.43	0.372
TKN	mg/l	19	0.82	5.1	1.8	0.945
BOD	mg/l	9	2	12	5	2.896
COD	mg/l	13	25	104	60	24.74
Lead	mg/l	12	0.044	0.280	0.12	0.079
Copper	mg/l	6	0.007	0.040	0.018	0.012

TABLE V-12 (Continued)

Summary of Event Characteristics
Station 9 Upstream Pipe Outlet

Parameter	Median Conc.	Antilog 10 Mean	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	98	103	2.0135	0.3614
TP	0.32	0.35	-0.45682	0.24889
TKN	1.6	1.6	0.21696	0.18725
BOD	5	5	0.66665	0.21795
COD	63	56	1.7465	0.19499
Lead	0.09	0.10	-0.99966	0.26472
Copper	0.011	0.015	-1.8147	0.28628

TABLE V-13

Summary of Event Characteristics
Station 8 Downstream Pipe Inlet

Parameter	n	Min.	Max.	Mean	Standard Deviation
Rainfall (inches)	23	0.14	1.05	0.55	0.23357
Runoff Volume (ft.^{3})	23	12,000	135,000	39,700	27,150
Runoff Coefficient	23	0.05	0.35	0.12	0.06032
TSS mg/l	23	43	679	170	143
TP mg/l	23	0.13	1.21	0.40	0.2223
TKN mg/l	22	0.76	6.3	2.0	1.1143
BOD mg/l	11	2	31	8	7.8297
COD mg/l	15	40	194	71	39.22
Lead mg/l	18	0.022	0.923	0.177	0.2459
Copper mg/l	8	0.005	0.119	0.026	0.0379

TABLE V-13 (Continued)

Summary of Event Characteristics
Station 8 Downstream Pipe Inlet

Parameter	Median Conc.	Antilog 10 Mean	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	122	130	2.1152	0.32162
TP	0.37	0.35	-0.45828	0.22366
TKN	2.0	1.8	0.26012	0.19734
BOD	6	7	0.81360	0.30366
COD	60	64	1.8075	0.18737
Lead	0.086	0.096	-1.0175	0.45988
Copper	0.013	0.016	-1.8034	0.40336

TABLE V-14

Summary of Event Characteristics
Station 7 Downstream Pipe Outlet

Parameter		n	Min.	Max.	Mean	Standard Deviation
Rainfall (inches)		21	0.34	0.96	0.59	0.195
Runoff Volume (ft.^{3})		21	12,000	135,000	44·181	29,989
Runoff Coefficient		21	0.05	0.39	0.12	0.062
TSS	mg/l	21	42	387	175	96.38
TP	mg/l	20	0.11	1.41	0.43	0.265
TKN	mg/l	20	1.0	4.4	1.8	0.720
BOD	mg/l	12	3	59	10	15.46
COD	mg/l	15	0.43	90	51	29.11
Lead	mg/l	14	0.023	0.208	0.10	0.0528
Copper	mg/l	10	0.009	0.208	0.034	0.061

TABLE V-14 (Continued)

Summary of Event Characteristics
Station 7 Downstream Pipe Outlet

Parameter	Median Conc.	Antilog 10 Mean	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	171	147	2.1679	0.28380
TP	0.39	0.37	-0.42888	0.23342
TKN	1.6	1.7	0.23259	0.13840
BOD	5	7	0.83694	0.33595
COD	58	31	1.4880	0.66612
Lead	0.087	0.080	-1.0994	0.29828
Copper	0.012	0.019	-1.7292	0.39137

TABLE V-15

Linear Regression Statistics
Station 10 Upstream Pipe Inlet

Dependent Variable	Independent Variable	n	Correlation Coefficient	Standard Error	F Statistic	Significance*
Runoff Coefficient	Total Rain	20	0.07	0.046	0.095	0.7612
TSS Concentration	Total Rain	20	0.11	154.3	0.239	0.6311
TSS Load	Total Rain	20	0.62	192.5	11.51	0.0032
TP Concentration	Total Rain	17	0.08	0.373	0.10	0.7578
TP Load	Total Rain	17	0.09	3.13	0.147	0.7069
TKN Concentration	Total Rain	16	0.19	0.923	0.507	0.4882
TKN Load	Total Rain	16	0.64	1.1	9.47	0.0082
BOD Concentration	Total Rain	9	0.59	1.45	3.702	0.0957
BOD Load	Total Rain	9	0.57	2.47	3.402	0.1076
COD Concentration	Total Rain	11	0.16	34.25	0.236	0.6389
COD Load	Total Rain	11	0.79	22.3	15.42	0.0035
Lead Concentration	Total Rain	13	0.20	0.087	0.478	0.5037
Lead Load	Total Rain	13	0.78	0.052	16.65	0.0018
Copper Concentration	Total Rain	7	0.02	0.157	0.028	0.9598
Copper Load	Total Rain	7	0.11	0.075	0.058	0.8118

* See Statistical Analysis for a discussion of significance.

TABLE V-16

Linear Regression Statistics
Station 9 Upstream Pipe Outlet

Dependent Variable	Independent Variable	n	Correlation Coefficient	Standard Error	F Statistic	Significance*
TSS Concentration	Total Rain	22	0.52	105.41	7.35	0.0134
TSS Load	Total Rain	22	0.86	147.90	56.86	0.0000
TP Concentration	Total Rain	19	0.14	0.38	0.35	0.56
TP Load	Total Rain	19	0.66	0.32	13.11	0.0021
TKN Concentration	Total Rain	19	0.0085	0.97	0.0012	0.9726
TKN Load	Total Rain	19	0.71	1.01	17.46	0.0006
BOD Concentration	Total Rain	9	0.24	3.00	0.44	0.5294
BOD Load	Total Rain	9	0.84	2.10	16.34	0.0049
COD Concentration	Total Rain	13	0.02	25.83	0.0034	0.9545
COD Load	Total Rain	13	0.91	19.76	51.90	0.0000
Lead Concentration	Total Rain	12	0.02	0.083	0.0031	0.9567
Lead Load	Total Rain	12	0.64	0.038	6.83	0.0259
Copper Concentration	Total Rain	6	0.43	0.012	0.90	0.3965
Copper Load	Total Rain	6	0.46	0.0065	1.10	0.3527

* See Statistical Analysis for a discussion of significance.

TABLE V-17

Linear Regression Statistics
Station 8 Downstream Pipe Inlet

Dependent Variable	Independent Variable	n	Correlation Coefficient	Standard Error	F Statistic	Significance*
Runoff Coefficient	Total Rain	23	0.04	0.062	0.034	0.8560
TSS Concentration	Total Rain	23	0.12	145.34	0.30	0.5912
TSS Load	Total Rain	23	0.46	351.90	5.77	0.0256
TP Concentration	Total Rain	23	0.02	0.23	0.0057	0.9404
TP Load	Total Rain	23	0.51	0.56	7.35	0.0131
TKN Concentration	Total Rain	22	0.0056	1.14	0.63	0.9802
TKN Load	Total Rain	22	0.54	3.06	8.07	0.0101
BOD Concentration	Total Rain	11	0.06	8.24	0.033	0.8601
BOD Load	Total Rain	11	0.19	17.68	0.35	0.5708
COD Concentration	Total Rain	15	0.06	40.62	0.052	0.8233
COD Load	Total Rain	15	0.46	89.84	3.43	0.0869
Lead Concentration	Total Rain	18	0.034	0.25	0.018	0.8945
Lead Load	Total Rain	18	0.11	0.75	0.21	0.6496
Copper Concentration	Total Rain	8	0.09	0.041	0.051	0.8292
Copper Load	Total Rain	8	0.20	0.14	0.25	0.6324

* See Statistical Analysis for a discussion of significance.

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TABLE V-18

Linear Regression Statistics
Station 7 Downstream Pipe Outlet

Dependent Variable	Independent Variable	n	Correlation Coefficient	Standard Error	F Statistic	Significance*
TSS Concentration	Total Rain	21	0.31	93.95	2.05	0.1686
TSS Load	Total Rain	21	0.70	252.34	18.61	0.0004
TP Concentration	Total Rain	20	0.14	0.27	0.35	0.5621
TP Load	Total Rain	20	0.56	0.70	8.06	0.0109
TKN Concentration	Total Rain	20	0.09	0.74	0.15	0.7026
TKN Load	Total Rain	20	0.64	2.60	12.24	0.0026
BOD Concentration	Total Rain	12	0.30	15.50	0.96	0.3493
BOD Load	Total Rain	12	0.38	53.87	1.69	0.2227
COD Concentration	Total Rain	15	0.42	27.41	2.78	0.1193
COD Load	Total Rain	15	0.23	98.59	0.74	0.4053
Lead Concentration	Total Rain	14	0.32	0.052	1.35	0.2673
Lead Load	Total Rain	14	0.68	0.16	10.33	0.0074
Copper Concentration	Total Rain	10	0.095	0.065	0.073	0.7932
Copper Load	Total Rain	10	0.02	0.22	0.0032	0.9562

* See Statistical Analysis for a discussion of significance.

A linear regression indicated that there was no statistically significant linear relationship between TSS concentration and rainfall at the upstream pipe (Table V-15) inlet. TSS concentration is not adequately described by either a log-normal or a Gaussian distribution, although log-normal distribution is assumed (Table V-11 and V-12). A linear regression analysis of suspended solids load indicated a relatively high correlation to rainfall at the upstream pipe (Tables V-15 and V-16). Removal of suspended solids is related to the particle size distribution and detention time. The measured particle size distribution as percentages of the total for the study period is presented in Table V-19. This data indicates a shift in particle size distribution between the inlet and outlet of the upstream pipe. It should also be noted that a relatively small percentage of the total is in the greater than 62.5 micron range.

The downstream upsize pipe displayed relatively low removals on a straight percentage basis (net export to a maximum of 36% removal). Net export occurred during 4 of 16 events. A mass balance for all events over the study period indicated no removal of the suspended solids input of 7,000 pounds by the downstream pipe.

A linear regression analysis of TSS concentration and load indicated that there are no statistically significant relationships to rainfall, except at the pipe outlet. The downstream pipe outlet had a correlation coefficient of 0.70 at the 99 percent significance for the linear regression of

TABLE V-19

Particle Size Distribution
Upstream Upsize Pipe

Size (micron)	Inlet %	Outlet %
>125	3	1
125-62.5	12	1
62.5-10	42	49
10-4	20	27
4-1	17	18
<1	6	4

TABLE V-20

Particle Size Distribution
Downstream Upsize Pipe

Size (micron)	Inlet %	Outlet %
>125	3	1
125-62.5	1	7
62.5-10	32	36
10-4	30	28
4-1	25	19
<1	9	9

TSS load against total rain (Table V-17 and V-18). The explanation for such a correlation is unknown. The distribution of TSS concentration at the downstream unit is not adequately described by either log-normal or Gaussian distribution. Log-normal distribution is assumed (Table V-13 and V-14). Particle size distribution, as a percentage of the study period totals, is presented in Table V-20.

The particle size distribution at the downstream unit does not show the shift in size between the inlet and outlet as was observed for the upstream pipe. However, the low percentage of particles in the greater than 62.5 micron range is similar.

Total Phosphorus

On an event basis, TP removal was highly variable at both upsize pipes. Treatment efficiency ranged from net export to 40 percent removal at both pipes. A mass balance for all events monitored during the study indicated no overall removal of TP by either pipe. The total influent loads were 7.5 pounds and 15 pounds for the upstream and downstream pipe, respectively.

A linear regression of TP concentration against total rainfall indicated that there is no statistical significant relationship at either the inlet or outlet for both pipes (Tables V-15 through V-19). Total phosphorus concentration at both pipes appears to be log-normal in distribution

(Tables V-11 through V-14). There appears to be very little relationship of TP load to rainfall at the upstream pipe inlet (r=0.08). However, the other three stations monitored in the system displayed somewhat weak relations of TP load to rainfall (Tables V-11 through V-14).

Total Kjeldahl Nitrogen

Total Kjeldahl nitrogen (TKN) displays the same type of variability as TP. Event removal of TKN ranged from net export to 30% removal at both pipes. A study period mass balance showed no removal of an influent load of 30 pounds at the upstream pipe and no removal of 70 pounds at the downstream pipe.

Linear regressions of TKN concentration and load against rainfall were performed. TKN concentration shows no significant statistical relation to rainfall at any of the four stations monitored in the system. TKN load displayed a weak but significant relationship to rainfall at all four monitoring stations (Tables V-15 through 18). TKN concentration can be described equally well by either log-normal or Gaussian distributions, although a log-normal distribution is assumed (Tables V-11 through V-14).

Biochemical Oxygen Demand

Biochemical oxygen demand (BOD) typically showed little or no removal on an event basis. Export of BOD was frequent although removal of up to 30 to 40 percent of influent mass was observed. A study period mass balance indicated no removal of BOD at either pipe. BOD input over the study period to the upstream pipe was 60 pounds and to the downstream pipe was 195 pounds.

BOD load and concentration displayed weak but significant relationships to rainfall at the upstream pipe, with the exception of concentraton at the outlet (Station 9). This data, however, should be viewed with caution because of the relatively small same size (n=9). BOD concentration and load had no statistically significant relation to rainfall at the downstream pipe. Again, however, the small sample size prohibits a high degree of confidence in the analysis (Tables V-15 through V-18).

BOD concentration, because of the small sample size, can be described equally well by either log-normal or Gaussian distribution. Log-normal distribution is assumed when the data is used elsewhere in this report.

Chemical Oxygen Demand

Event removal of COD was generally very low. Export occurred frequently and maximum observed removal was 40%. A mass balance of all monitored events indicated no removal of COD at either upsize pipe. Influent mass input to the upstream and downstream pipes was 600 pounds and 1,800 pounds, respectively, over the duration of the study.

Linear regression analyses of COD load versus rainfall indicated a significant relationship exists at the upstream pipe, with the outlet displaying a higher correlation than the inlet (Tables V-15 and V-16). The linear regression of COD load against rainfall at the downstream pipe indicated no significant relation (Tables V-17 and V-18). The linear regression of COD concentration versus rainfall indicated no correlation exists at any of the four stations monitored (Tables V-15 through V-18).

Concentration of COD is not well described by either normal or log-normal distributions. For the purposes of further analysis log-normal distribution is assumed.

Lead

As with the other parameters discussed above, lead removal is highly variable. However lead is removed more consistently than the nutrients and is not frequently exported. The study period mass balance indicated that the upstream pipe removed 25% of an influent mass of 1.3 pounds and the downstream pipe removed 50% of an influent mass of 6.4 pounds.

A linear regression of lead concentration against rainfall showed no significant relationship at any of the monitored stations. However, the upstream pipe displayed a significant relation of lead load to rainfall. The downstream pipe inlet analysis indicated that lead load was unrelated to rainfall while the outlet displays a correlation of 68 percent at the 99 percent confidence level.

Lead concentration is likely best described by a log-normal distribution and is assumed to be so for calculation of land use relationships. The distribution is not obviously log-normal however and may be equally well described by an arithmetic distribution.

Copper

Copper removal displayed very little consistency on an event basis. Typically, when removal occurred it was low, (in the 10-20% range) although removals in the 90% range were observed in the upstream pipe. A study period mass balance indicated that the upstream pipe removed 60% of the influent mass of 0.3 pounds while the downstream pipe removed none of an influent mass of 0.7 pounds.

Linear regression analyses of copper load and copper concentration against rainfall indicated no correlation exists at any of the four monitoring stations. The data should be used with caution however because of the small sample sizes involved.

A log-normal distribution of copper concentration is assumed. However, because of the small number of samples in the analysis, the distribution is not clearly defined.

Accumulated Sediment

As part of the program for evaluating the performance of the upsize pipe system, the pipes were emptied of water and the amount of sediment measured and sediment samples collected. The sediment was measured manually on three occasions - 9/15/80, 11/3/81, and 4/14/82. Sediment samples were taken on 9/15/80 and 11/3/81. Sediment composition is presented in Table V-21. In addition, the pipes were cleaned of accumulated sediment in June of 1981.

The sediment depth measurements were converted to volume using the geometric elements for circular channel sections. Volumes and mass in each pipe as measured on 9/15/80 and 11/3/81 are given in Table V-21. The measured volume on 4/14/82 was 170 cubic feet in the upstream pipe and 65 cubic feet in the downstream pipe. The mass using 1.7 gm/ml for the upstream pipe and 1.6 gm/ml for the downstream pipe is 18,000 pounds and 6,500 pounds, respectively.

Both upsize pipes were cleaned in early June of 1981. A combined total of 380-430 cubic feet (wet volume) was removed. Therefore, from June to November, 1981 the upstream pipe accumulated approximately 4,200 pounds of sediment and the downstream pipe collected approximately 5,000 pounds.

TABLE V-21

Sediment Composition
In-Line Upsize Pipes

Parameter		September, 1980 Upstream Pipe	September, 1980 Downstream Pipe	November, 1981 Upstream Pipe	November, 1981 Downstream Pipe
Total Solids %		70	60	71	60
Total Volatile Solids %		3.3	7.4	3.1	3.1
Density	gm/ml	1.7	1.6	1.7	1.6
TKN	mg/kg	430	740	680	700
NO_3+NO_2-N	mg/kg	3	3	19	12
TP	mg/kg	300	640	410	700
TOC	mg/kg			22,000	32,000
COD	mg/kg	41,000	84,000	66,000	44,000
Lead	mg/kg	160	175	230	140
Iron	mg/kg	8,700	13,000	12,000	10,000
Zinc	mg/kg	65	200	200	260
Chromium	mg/kg	30	20	2.4	2.3
Copper	mg/kg	19	54	24	85
Nickel	mg/kg	15	20	1.8	3.2
Cadmium	mg/kg	1.5	3	2.3	2.6
Sediment Volume	$ft.^3$	90	100	40	50
Mass	lbs.	9,600	9,000	4,200	5,000

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Discussion

As noted in the discussion of removal efficiency, a mass balance including all monitored events indicated little or no removal of pollutants at either upsize pipe. However, visual inspecton of the pipe sumps and the cleaning of the pipes indicated substantial amounts of sediment had accumulated. The following will discuss this obvious discrepancy. The discussion will key on the upstream pipe, however the items mentioned will apply to the downstream pipe as well.

During the five month period after the pipes were cleaned, the upstream pipe accumulated approximately 4,200 pounds of sediment. During that period 20 precipitation events with a depth greater than 0.1 inches occurred, seven of which were monitored. Eight of the 20 storms were less than 0.25 inches. During the total study period 20 events were monitored at the inlet with a mean depth of 0.59 inches and a range between 0.17 inch and 1.63 inch. Total measured input load during the study period was 4,000 pounds.

An attempt was made to relate observed mass loading to the events which occurred during the five month period between the cleaning and final visual inspection. Storms with less than 0.3 inches typically had a suspended solids load from 50-150 pounds. Storms from 0.3 to 1 inch typically had TSS loads of 100-500 pounds. Therefore, assuming eight storms with an average load of 100 pounds and

12 storms with 300 a pound load, the total inlet load would have been approximately 4,400 pounds. Hence, to account for the amount of solids in the pipe - using observed mass load rates - average removal efficiency would have to be close to 100%.

A possible explanation for the discrepancy in removal efficiency predicted by the monitoring program and the accumulated material observed in the pipe is that the sampling protocol was missing the larger sand-like materials. Visual inspections of the pipe sump revealed a large pile of sandy material near the inlet of the pipe. It was found that the pile of sand comprised 40 - 50 percent of the total mass in the pipe. The remaining material was made up of finer, clay-organic type material. Sand type material usually rolls along the pipe invert and constitutes the bed load of the drainage system.

In order to obtain a representative sample of all constituents, sampling ports were placed 1-2 inches above the pipe invert. Although the samplers used were of the suction, quick lift type and were selected because of their ability to draw the heavier particles over a long lift length, it is possible the sand was not entering the suction zone of the sampling port.

Early in the study it was noted that the sampling protocol may not have been effective in picking up some portion of the inlet load. The QA program included grab samples in the immediate area of the sampling port while the sampler was taking a sample. The QA data indicates that the sampling equipment was effective in sampling the area around the screen. However, QA grab samples would probably not be any more effective in sampling bed load than the suction lift samplers. It is nearly impossible to hold a sample bottle directly on the bottom of a storm sewer at high flow - in fact at some stations it would have been extremely dangerous to even attempt such sampling.

A mass balance over the entire study period for both units show little or no removal of the various nutrients and metals, with some exceptions. However, on an event basis nutrients and metals were removed by the upsize pipe system, although as noted in the discussion on removal efficiency, these removals were highly variable.

The data suggests that small events tend to display good removal of nutrients and metals while larger events tend toward lower removals of these materials. Small storms and large storms of long duration result in low velocities and long detention time. Low velocity tends to minimize scour and long detention time facilitates removal of small particle sizes. However, removal and export is so variable that no statistically significant trends could be identified - large high intensity storms occasionally displayed relatively high removals of pollutants while small events periodically resulted in export of nearly every parameter method.

Analysis of accumulated sediment clearly indicates that removal of nutrients and metals is occurring, even though a mass balance of runoff samples for all events indicate that little or no removal is taking place. However, the mass balance removal efficiency is misleading, because typically only the larger storms generated enough flow to monitor. Generally speaking, approximately 6,000-10,000 cubic feet of runoff volume was required to generate enough sample volume for analysis. Given the mean runoff coefficient of 0.12, this volume translates into 0.2 to 0.3 inches of rain, minimum, depending of course on duration, intensity, and antecedent dry days. The mean monitored event size was 0.59 inches and this size storm generally showed poor removals of nutrients and metals.

Rainfall data collected during the study period show numerous storms in the 0.1 - 0.3 inch range. These small storms contribute to the overall accumulation of solids, nutrients, and metals in the upsize pipe system. The data, however, do not allow any estimation of load or removal efficiency during these events, and therefore the amount contributed by the unmonitored events is unknown.

As noted above, it is felt that unmonitored bed load of heavier inert material contributes a large portion of the accumulated solids in the upsize pipe system. It would be expected then that the proportion of nutrients and metals in the accumulated solids should be less than that seen in the runoff, due to the large share of sand like material observed in the pipe sump. The hypothesis is supported by the fact that the total phosphorus constitutes 0.02 to 0.04 percent of the solids in the accumulated sediment (Table V-21), while in runoff TP is usually 0.11 to 0.43 percent of the solids load.

As stated in the objectives for this study, it was anticipated that removal of pollutants in the upsize pipes would be accomplished primarily by the sedimentation process. The principal factor under ideal conditions which affects sedimentation efficiency is detention time. In the upsize pipe system detention time varies with velocity and is a function of total rainfall and intensity. Average detention times ranged from 20 to 120 minutes in the upstream pipe and from 9 to 70 minutes in the downstream pipe. Minimum detention time will be signficantly less than average detention time during peak storm intensity. It should also be noted that minimum detention time will occur when the greatest load of material is expected to be entering the system.

The effect of velocity on removal efficiency of TSS was examined. The data indicated that there is no statistically significant relationship between removal of TSS and velocity at either upsize pipe.

Costs

Cost data for the in-line upsized pipes are presented in Tables V-22 and V-23. The cost of the installation

TABLE V-22

Cost Data for the Upstream In-line Upsized Pipe

Capital Cost (in 1980 Dollars)

Upsized Pipe (144'L, 96"D)	$48,900.00
Manholes	6.000.00
Replace Pavement	3,100.00
	$58,000.00
Engineering Design and Inspection	$6,500.00
Review and Approval of Design Plans	1,200.00
Legal and Financial Fees	600.00
Contingency	3,900.00
	$12,200.00
Total Installed Capital Cost	$70,200.00

Annual Operation Cost (in 1980 Dollars)

Routine Maintenance	$240.00
Waste Disposal	2,550.00
Capital Cost*	3.510.00
Interest**	4,171.00
Total Annual Operating Cost	$10,471.00

* Straight-line depreciation of total installed capital cost over 20 years.

** Bonds at 5.9419%. principal of $70,200.00.

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TABLE V-23

Cost Data for the Downstream In-line Upsized Pipe

Capital Cost (in 1980 Dollars)

Upsized Pipe (85'L, 96"D)	$30.100.00
Manholes	6.000.00
Replace Pavement	1,900.00
	$38,000.00
Engineering Design and Inspection	4,300.00
Review and Approval of Design Plans	1,200.00
Legal and Financial Fees	400.00
Contingency	2,600.00
	$8,500.00
Total Installed Capital Cost	$46,500.00

Annual Operating Cost (in 1980 Dollars)

Routine Maintenance	$240.00
Waste Disposal	1,500.00
Capital Cost*	2,325.00
Interest**	2,763.00
Total Annual Operating Cost	$6,828.00

* Straight-line depreciation of total installed capital cost over 20 years.

** Bonds at 5.9419%. principal of $46.500.00

includes excavation, placing the pipe, backfilling with the appropriate materials and compacting, and hauling the excess excavated material to a dump site one mile away, in addition to the cost of the pipes themselves. These costs were based on cost factors found in the 1980 Dodge Guide.[4] The costs for the manholes and pavement replacement were taken from the contractors bid documents. Engineering design and inspection fees were assumed to be 11.2% of the construction cost, legal and financial fees were assumed to be 1% of the construction cost, and the contingency costs were assumed to be 6.8% of the construction cost. The costs for design review and approval were based on estimates of the time requirements of drain commission personnel. Since these BMP's are located beneath a city street there were no right-of-way or site acquisition costs.

Routine maintenance costs assume that four inspections of each BMP will be performed by drain commission personnel every year, at a cost of $60.00 per inspection. The cost for waste disposal assumes the upsized pipes will be cleaned twice a year using a vacuum truck. The cost of cleaning the pipes, and the volumes of sediment to be removed, were calculated from the actual cleaning of the BMP's in June of 1981. Dumping fees of $4.00 per cubic yard of material were used. This assumes that the sediment will not have to be handled as a hazardous waste.

The annual capital cost was determined from straight-line depreciation of the total installed capital cost over a twenty year period. Annual interest costs assumed an interest rate of 5.9419% (the actual interest rate of the bonds sold to finance the Bogus Swamp Drain project) and a principal equal to the total installed capital cost.

OFF-LINE DETENTION BASIN

Introduction

A description of land use and station set-up can be found in Sections III and IV. The off-line detention basin was designed to provide flood relief for downstream areas during storms larger than the one year storm. According to design specifications, the basin would operate when storm generated flow exceeded the discharge capacity of the downstream pipe. The water would then flow out of a beehive opening and into an undeveloped grassy area. As a pollution control device it was anticipated the basin would act as a settling basin whenever the area was flooded.

After initiation of the flow monitoring program it became apparent that the basin was not receiving sufficient flow to flood the basin with any reasonable frequency. Therefore, an inflatable pillow was installed in the outlet to provide further restriction of outlet flow. The pillow was inflated when storm events of sufficient size were forecast (>0.5 inches). In order to simulate a steady outflow, lengths of pipe were insearted under the pillow, allowing water to be discharged at a decreased steady rate.

Data Collection

As described earlier in this report, monitoring of the basin consisted of determination of flow and pollutant levels at the inlet and outlet stations. At the inlet a Manning level recorder and manual velocity measurements were used to calculate flow. At the outlet a Manning sonar unit, programmed with a site specific stage-discharge curve, measured discharge. Samples were taken by automatic discrete samplers, activated at set time increments at the inlet and linked to the flow meter at the outlet.

The methods of flow measurement mentioned above were required by site specific conditions. At both monitoring stations large amounts of sand and grit were deposited in the pipe inverts precluding the use of the Manning Formula. Thus, site specific stage-discharge relationships were determined for each station, so that the discharge could be determined directly from the measured depth of flow.

The basin was monitored twice during the study period. The first monitored event was a 2.82 inch rain that began on May 10, 1981. This event was a discrete sample survey. The second event occurred on August 27, 1981, when flow weighted composite samples were taken during an 0.86 inch rain. The outlet station (station 3) was monitored during six additional events to increase the source loading data base being obtained during the study. During these six additional events flow passed straight through the pipe network with no flooding of the detention basin.

During both monitored events the flows balanced only within approximately 20 percent between the inlet and outlet stations. During the May event, inlet flow volume was estimated at 519,000 cubic feet and the outlet at 704,000 cubic feet. During the August event inlet flow volume was estimated to be 65,000 cubic feet and outflow at 52,000 cubic feet. While undesirable, this type of imbalance can be somewhat expected due to site specific conditions and available measuring techniques. Flow was particularly difficult to determine at the inlet station due to backwater effects from the downstream restriction.

The inlet and outlet stations were both characterized by extensive sediment depositions in the pipe inverts. Flow was determined by measuring the depth of sediment before and after events and averaging these values to obtain an average cross sectional area. Then, during the course of the event, velocity and total depth of flow measurements were made periodically. To calculate flow, the average sediment area was subtracted from the total "wetted" cross sectional area to obtain cross sectional area of flow. The effective cross sectional area was then multiplied by velocity to obtain total discharge. Obviously, this technique presents a great margin for error, especially when one considers the need to read a hand held velocity meter under extremely adverse weather conditions.

Results

During May 1981 a discrete sample survey was performed. Runoff, generated by a 2.82 inch rain, lasted for 57 hours. At the start of the survey baseflow samples were taken and the pillow inflated. Twenty seven hours into the survey an intense period of rain occurred. During this time the pillow had to be deflated to increase the outflow rate and control the pond level, as it was feared that any additional water entering the basin would flood local basements. After the basin was sufficiently lowered and the heavy spate had abated, the stopper was reinflated. The outlet continued to be restricted and was held at a constant flow until the basin was empty. Based on influent and effluent volumes, observed depth of the water column, and basin geometry, it is estimated that the basin treated 120,000 cubic feet of water or approximately 20 percent of the runoff water passing through the drain. The maximum detention for the water treated during the event was approximately 25 hours and the minimum approximately 15 hours. Maximum pond depth was 2-3 feet.

Examination of the discrete survey concentration data provides little evidence of the first flush phenomenon. Concentration of all parameters, with the exception of TSS, remained relatively flat over the course of the survey (Table V-24 and V-25). Total suspended solids concentration displayed a sharp increase in both the inlet and outlet samples when the basin was lowered in the middle of the event. The fact that none of the other parameters show as

TABLE V-24

Discrete Survey
Station 4 Basin Inlet

Sample #	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Flow cfs	0.23	1.0	1.2	2.1	2.4	2.4	2.4	2.7	4.7	5.3	3.2	2.7	3.8	3.8
TS Concentration mg/l	950	390	540	470	420	440	470	380	210	330	380	380	390	360
TSS Concentration mg/l	5	20	15	12	40	13	6	24	60	21	28	40	18	12
TP Concentration mg/l	0.2	0.15	0.07	0.07	0.12	0.11	0.07	0.09	0.26	0.18	0.14	0.11	0.22	0.11
TKN Concentration mg/l	1.2	1.6	1.2	1.3	1.2	1.0	1.2	1.0	1.2	1.7	1.4	1.2	1.6	1.2
SKN Concentration mg/l	1.1	2.1	0.82	0.82	0.91	0.87	0.76	0.68	0.68	0.94	1.8	0.86	1.0	8.74
BOD Concentration mg/l						4		3		5	4		7	
COD Concentration mg/l	21	39	37	30	38	26	30	21	30	30	23	26	27	26
Lead Concentration mg/l	0.002	0.074	0.012	0.030	0.095	0.022	0.012	0.013	0.037	0.015	0.022	0.013	0.013	0.007
Copper Concentration mg/l	0.010	0.010	<0.010	<0.010	0.020	0.010	<0.010	0.010	<0.010	0.010	<0.010	<0.010	<0.010	<0.010

TABLE V-25

Discrete Survey
Station 3 Basin Outlet

Sample #	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Flow cfs	0.23	0.75	1.0	0.9	1.0	2.2	3.2	2.5	10.9	10.9	3.0	11.1	6.1	4.3
TS Concentration mg/l	950	260	380	480	510	420	380	360	270	330	390	360	480	400
TSS Concentration mg/l	5	28	14	4	12	13	10	21	120	10	15	15	34	14
TP Concentration mg/l	0.20	0.21	0.09	0.08	0.08	0.06	0.09	0.08	0.23	0.1	0.11	0.11	0.17	0.11
TKN Concentration mg/l	1.2	2.2	1.0	1.1	1.3	1.1	1.2	0.95	1.5	1.1	1.2	1.1	1.6	1.3
SKN Concentration mg/l	1.1	1.6	0.87	1.2	0.82	0.83	0.94	0.70	0.52	0.96	0.86	0.77	1.1	0.99
BOD Concentration mg/l						5		2		3	3		5	
COD Concentration mg/l	21	55	35	31	34	30	30	25	30	25	26	30	32	33
Lead Concentration mg/l	0.002	0.050	0.030	0.017	0.024	0.025	0.017	0.012	0.014	0.015	0.006	0.012	0.026	0.007
Copper Concentration mg/l	0.010	<0.010	<0.010	<0.010	0.010	<0.010	0.010	<0.010	0.010	<0.010	<0.010	<0.010	<0.010	0.010

sharp an increase as TSS suggests that the increase may be due to scouring of the sandlike materials in the bed load. Since concentration remains relatively flat and the above mentioned increase shows up in only one sample at each station, calculation of event mean concentration using the flow measurements should be representative of conditions in the basin, regardless of any error in flow determination. Based on the event mean concentration the off-line detention basin had very little effect on pollutant levels. The parameters of interest show little or no change in concentration between inlet and outlet (Table V-26).

During the August, 1981 event, composite samples were taken of runoff generated by a 0.86 inch rain. The runoff lasted for 10 hours. Based on influent and effluent flow, approximately 34,000 cubic feet of water was treated, with an average detention time of 5 hours. The basin treated about 65% of the total flow as calculated from the effluent discharge volume. As was true with the May survey, the basin provided very little, if any, treatment of the runoff (Table V-27).

As noted above, the basin outlet (Station 3) was monitored during six additional events for determining pollutant loading levels in this portion of the district. A summary of event characteristics is given in Table V-28. This table presents minimum, maximum, mean, and other statistics for all events monitored at the station, including the two events when the basin was flooded. If a log normal distribution of concentration is assumed, the

TABLE V-26

Comparison of Events
Off-Line Detention Basin

	5/10/81 - 5/12/81		8/27/81	
Parameter	Inlet Station 4	Outlet Station 3	Inlet Station 4	Outlet Station 3
Runoff Volume	519,000	704,000	65,000	52,000
Total Solids Concentration	360	370	310	330
TSS Concentration	19	22	25	35
TP Concentration	0.14	0.13	0.10	0.15
TKN Concentration	1.3	1.3	1.2	1.7
Soluble Kjeldahl Concentration	0.90	0.87	1.1	1.3
BOD Concentration	5	4	4	4
COD Concentration	28	30	31	38
Lead Concentration	0.02	0.012	0.034	0.04
Copper Concentration	<0.01	<0.01	0.01	0.01
Particle Size (micron)				
>125 %	0	0	0	7
125-62.5 %	0	0	13	0
62.5-10 %	39	36	27	34
10-4 %	17	23	20	19
4-1 %	28	27	23	29
<1 %	17	14	17	11

TABLE V- 27

Summary of Event Characteristics
Station 3 Off-Line Detention Basin

Parameter	n	Min.	Max.	Mean	Standard Deviation
Rainfall inches	8	0.32	2.82	0.93	0.798
Runoff Volume (ft.^{3})	8	6,000	704,000	126,000	235,200
Runoff Coefficient	8	0.03	0.53	0.18	0.153
TSS mg/l	7	13	200	66	67
TP mg/l	8	0.08	0.27	0.17	0.076
TKN mg/l	7	0.65	1.69	1.3	0.377
BOD mg/l	4	2	6	4	1.63
COD mg/l	5	30	60	39	12.2
Lead mg/l	7	0.012	0.187	0.079	0.07
Copper mg/l	5	0.005	0.010	0.007	0.003

TABLE V-27 (Continued)

Summary of Event Characteristics
Station 3 Basin Off-Line Detention Basin

Parameter	Median Conc.	Antilog 10 Mean	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	34	43	1.6380	0.43432
TP	0.15	0.16	-0.80564	0.20960
TKN	1.3	1.2	0.08007	0.14741
BOD	4	4	0.57052	0.20104
COD	37	38	1.5767	0.12283
Lead	0.04	0.05	-1.2765	0.43723
Copper	0.005	0.006	-2.1886	0.15706

geometric mean TSS concentration delivered to the basin is 43 mg/l. For the two events monitored while the basin was flooded the influent TSS concentrations were 19 and 25 mg/l; much lower than the predicted most probable concentration. The concentrations of other parameters (e.g. TP) are much closer to the predicted EMC values. The total suspended solids EMC for all events is lower than that seen at other BMP inlet stations (e.g., Station 6 mean EMC=53 mg/l; Station 10 mean EMC=184 mg/l). For the two events monitored in the basin, TSS concentrations were much lower than those observed at other BMP inlet stations. Particle size distribution for the two events also tends toward the lower ranges as compared to other BMP inlet stations. For example, during the May event 100 percent of the available particles are smaller than the 62.5 micron size as determined by the settling tests.

Discussion

The very low inlet concentration of TSS, and the large percentage of particles in the low end of the size distribution may in some degree be due to an anomaly in the sampling program (i.e., a different 0.86" storm could have just as easily provided higher concentrations than observed from this particular storm). However, the overall trend toward low concentration is undoubtedly due to the fact that the off-line detention basin is receiving a significant portion of its influent from the on-line retention basin which is located upstream. For example, during the August event the basin received 70 percent of its flow and 40 percent of its suspended solids load from the retention basin. Obviously, the detention basin then is receiving a large quantity of water treated very effectively by the upstream basin. Thus, a substantial portion of the water entering the dry basin was treated to the maximum attainable level by the sedimentation process alone (see discussion of on-line basin).

Data collected at the outlet of the on-line retention basin and the data collected at the inlet to the detention basin was used to calculate the loading of pollutants from the subdistrict area between the two stations (i.e. the "untreated" runoff water). This data, presented in Section VI, shows that the pollutant loadings per unit of drainage area were in the same range as the other subdistricts monitored. Therefore, the input loading from untreated water was typical of the drainage system. The log mean concentration of TSS for example was calculated to be 75 mg/l from the subdistrict. This concentration is in the range (52-145 mg/l) for the Bogus district as a whole, but is substantially lower than levels reported in the literature for many urban storm drains. This somewhat lower than expected level of pollutants for the untreated portion of the runoff water coupled with the high degree of control attained in the upstream basin, resulted in relatively low overall pollutant levels entering the dry basin area.

The dry basin detained only a portion of the influent water as was described earlier. The off-line design concept was based on diverting a portion of the flow to the basin during peak flow periods with a steady outflow of non-detained water occurring at all times. Using the May event as an example, only 20 percent of the water passing through the drain was diverted to the basin for passible treatment. Thus, assuming a 50% removal of TSS in the flooded portion of the basin, and an influent concentration of 21 mg/l (ave. of influent and effluent concentration) the change in observed effluent concentration would be less than 10%. Of course, given that a large share of the influent has already been treated, a removal efficiency of 50% is highly unlikely, making it even more difficult to detect any treatment occurring in the basin proper.

Another factor possibly contributing to the poor performance of the basin is detention time. During the August event, when approximately 65% of the flow was detained, average detention time was 5 hours. Based on ideal settling, particles with a diameter of 10 microns have a settling velocity of 0.8 ft./hr. in quiescent conditions. Using a pond depth of 2 feet, particles with a diameter greater than or equal to 10 microns should settle in 2.5 hours. Obviously, however, conditions in the basin are far from ideal, with wind effects, turbulence, and the like making the required detention time for settlement of such particles much greater. Once the particle has settled it must adhere to something to be removed from the water column. However, when the basin empties, water exits from the bottom. As the water drains out, velocities near the basin floor may approach scour velocity for these smaller particles, and they could become resuspended. In fact, if water velocity is high enough, some erosion could take place and the basin could actually contribute suspended solids. Some errosion effects were noted in the vicinity of the beehive inlet/outlet structure.

Given that the particle size distribution of the influent water tended toward the low end in all samples, the detention time in the basin was probably not long enough to provide any high degree of settling during the August event (5 hours average).

If it were assumed that 50% of the TSS concentration in the greater than 10 micron range could be removed (40% of the total TSS concentration was above 10 micron) with 65% of the total flow treated, the overall removal would be approximately 13%. Likewise, as shown earlier for the May event (20% of flow detained), the estimated overall removal was less than 10%. Thus, the combination of low suspended solids, small particles, and bypass flow effectively lowered the overall potential for removal in the basin to a point where it would be difficult to detect.

The site specific conditions found in this study resulted in a set of circumstances which make it difficult to determine if dry detention basins of this type could ever achieve significant control of pollutant loads. It is possible that if dry basins receive higher loadings of pollutants, particularly with larger suspended particle sizes, that some sedimentation would be achieved. However, observation of the off-line basin indicates that the problems of turbulence near the sediment-water interface during the draining of the basin and erosion near the outlet may result in poor performance due to resuspension of the settled particles.

This observation is further strengthened by visual inspection of several dry on-line basins in the southern Michigan area. In each case muddy bottom conditions and signs of erosion or channelization through these on-line basins is quite obvious. It is probable that with this type of condition occurring in these basins, performance for the control of pollutants is marginal to poor. It is also possible that these types of problems could be overcome by special design considerations during the planning phase. However, based on our observations, cooperation of the owners or inspection by the authorized public agency will be needed to insure that the appropriate design measures are utilized and installed properly. One noticeable problem in most dry basins is obtaining good sodding and/or bank stabilization at the end of construction. Without proper finishing details such as these, the dry basins exhibit severe erosion problems, particularly during the spring periods which are characterized by high flows and less well established plant growth.

Costs

Cost data for the Off-line Detention Basin is presented in Table V-20. Since no excavation or contouring of the existing basin was required the only improvement needed for the construction of the BMP was the installation of the pipe from the storm sewer to the basin. The cost of the pipe includes excavation, placing the pipe, backfilling with the appropriate material and compacting, fertilizing and reseeding the damaged area, and hauling the excess excavated material to a dump site one mile away. These costs were based on cost factors found in the 1980 Dodge Guide.[4] The cost for the manhole was taken from the contractors bid documents. Engineering design and inspection fees were assumed to be 11.2% of the construction cost, legal and financial fees were assumed to be 1% of the construction cost, and the contingency costs were assumed to be 6.8% of the construction costs. The cost of design review and approval was based on estimates of the time requirements of drain commission personnel.

There were no right-of-way or site acquisitions costs associated with this basin due to site specific conditions. A right-of-way existed for the currently installed pipe which runs through the basin. Originally, the flood relief project called for increasing the size of this pipe to alleviate flooding in the back yard areas along the main sewer line. However, the local residents indicated at public meetings that periodic flooding in the low areas of their yards (estimated at 1-4 times per year) was not a

TABLE V-28

Cost Data for the Off-Line Detention Basin

Capital Cost (in 1980 Dollars)

Inlet Pipe	$5,950.00
Manhole	800.00
	$6.750.00
Engineering Design and Inspection	$760.00
Review and Approval of Design Plans	600.00
Legal and Financial Fees	70.00
Contingency	460.00
	$1,890.00
Total Installed Capital Cost	$8,640.00

Annual Operating Cost (in 1980 Dollars)

Routine Maintenance	$240.00
Capital Cost*	432.00
Interest**	513.00
Total Annual Operating Cost	$1,185.00

* Straight-line depreciation of total installed capital cost over 20 years.

** Bonds at $5.9419%. principal of $8,640.00.

significant problem as long as the basements of their homes, which were on higher ground, did not flood. Based on this opinion, the new pipe was not installed saving considerable cost to the taxpayers. The result was the existence of the detention area in the low lying backyard portions of the private property with no land acquisition required for completion of the project.

Routine maintenace costs assume that the BMP will be inspected four times per year by drain commission personnel, at a cost of $60.00 per inspection. Since any sediment retained in the basin is incorporated into the lawn area, there are no associated waste disposal costs.

The annual capital cost was determined using straight-line depreciation of the total installed capital cost over a twenty year period. Annual interest costs assumed an interest rate of 5.9419% (the actual interest rate of the bonds sold to finance the Bogus Swamp Drain project) and a principal equal to the total installed capital cost.

SECTION VI

POLLUTANT SOURCES AND LOADS

INTRODUCTION

The driving force of urban runoff is rainfall/snowfall which occurs in the area. The precipitation data collected during this study is presented herein, in addition to an evaluation of storm characteristics based on thirty years of hourly rainfall records. A portion of this study included measurement of pollutant contribution originating from the atmosphere (wet and dry fall). This information is presented and related to the total urban runoff load. Finally, the data collected from the inlet stations to the BMP's has been used to calculate annual loads of various constituents entering the drains. This data has then been used to estimate the contributions of pollutants from the larger urban area.

PRECIPITATION

Rainfall intensity and volume were measured in the drainage district using a seven day recording weighing type gauge, described in Chapter IV. In addition, manual gauges were located at two locations in the district coinciding with the atmospheric sampling stations. These gauges also provided data on any significant variability in rainfall amounts which occurred across the basin during a given event. Generally there was very little difference in rainfall amounts within the 450 acre area. A backup recording rain gauge was maintained by the Lansing wastewater treatment plant located adjacent to the district outlet. Data from this station was used during times when malfunctions occurred in the primary recording gauge. The daily summaries for rainfall amounts during the project period are presented in Table VI-1. Detailed hourly rainfall records for this time period are presented in the separate Data Report.

Hourly precipitation data was obtained from the National Climate Center for Weather Service, station 204641, Lansing airport, for the period of record 1948-1978. This data was used to determine rainfall characteristics for the Lansing area. The data evaluation was accomplished using the SYNOP program. The University of Florida version of this program was obtained and adopted for use. All of the statistics presented here were computed using an inter-event time of five dry hours between storms. This delta time resulted in a coefficient of variation of 1.02 for the inter-event time. Shorter and longer time periods resulted in larger deviations from 1.0 in the coefficient of variation.

The SYNOP program calculates rainfall statistics on a monthly, yearly and by storm basis. Rainfall volume, intensity, duration and dry period duration are the parameters used to describe the rainfall patterns. The

TABLE VI-1

Precipitation Record - 1980
Daily Total (inches)

Day	March	April	May	June	July	August	Sept.	Oct.	Nov.
1	0	0	0	.43	0	0	0.28	0.07	0
2	0	.09	0	.21	0	0.39	0.05	0.04	0
3	0	.63	0	.52	0	0	0	0.23	0
4	0	.04	0	0	0	0	0	0	0.10
5	.06	0	0	.5	.92	0.19	0	0	0
6	0	0	0	.13	0	0	0	0	0
7	0	0	0	.89	0	0	0	0	0.21
8	.12	.23	0	0	0	0.10	0	0	0
9	0	.07	0	.06	.03	0	0.22	0	0.08
10	0	.03	0	0	0	0.53	0	0	0
11	0	.08	.1	0	0	0.17	0	0	0
12	0	0	0	0	0	0	0	0	0
13	.11	0	.68	0	0	0	0.07	0	0
14	0	.31	.02	.37	0	0	0.04	0.44	0.05
15	0	.11	0	.15	0	0	0	0	0
16	0	0	0	0	.96	0	0.73	0.31	0
17	.57	0	*.53	0	0	0.10	0.90	0.43	0
18	0	0	*.59	.04	0	0	0	0	0
19	0	0	0	.33	0	1.38	0	0	0
20	.05	0	0	0	0	1.56	0.08	0	0
21	.17	0	0	0	.3	0.20	0	0	0.02
22	0	0	0	0	0	0	0.49	0	0
23	0	0	0	0	0	0	0	0	0.02
24	.08	.26	0	0	0	0	0	0.43	0.11
25	0	.02	0	0	0	0	0.11	0	0
26	0	0	0	.02	.69	0	0	0	0
27	0	.06	0	0	1.08	0	0	0	0.18
28	.05	.35	0	.02	0	0	0	0	0.02
29	.1	.19	0	0	.09	0	0	0	0.08
30	.02	0	.03	0	0	0	0	0	0
31	.37	0	.25	0	.05	0.21	---	0	---
TOTAL	1.7	2.47	2.20	3.67	4.12	4.83	2.97	1.95	0.87

* Station 1 down
Wastewater Treatment Plant Gauge Used.

TABLE VI-1 (Continued)

Precipitation Record - 1981
Daily Totals (inches)

Day	Jan.	Feb.	March	April	May	June	July	August	Sept.	Oct.	Nov.
1	.03	0	0	0	0	0	0	0	.46	.62	0
2	.02	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	.08	0	.27	0	0
4	.01	0	0	.36	0	0	0	0	.83	0	0
5	0	0	0	0	0	0	0	0	0	.05	0
6	.11	0	0	0	0	0	0	0	0	.12	0
7	0	0	0	0	0	.57	0	0	.06	0	0
8	0	0	0	.25	0	.10	0	0	0	0	0
9	0	0	0	.41	0	.04	0	0	0	0	0
10	0	.75	0	.03	1.49	0	0	0	0	0	0
11	0	.06	0	1.24	1.33	0	0	0	0	0	0
12	0	0	0	.39	0	0	0	0	0	0	0
13	0	0	0	1.50	0	1.05	0	0	0	0	0
14	0	0	0	0	.09	.39	0	.06	0	.15	0
15	0	0	.11	0	.02	0	0	.04	0	.03	0
16	0	0	0	0	0	0	0	0	.55	0	0
17	0	0	0	0	0	0	0	0	.18	.14	0
18	0	0	0	0	0	0	0	0	0	.16	0
19	0	0	0	.11	0	0	0	0	0	0	.11
20	0	0	0	0	0	0	.17	0	0	0	.84
21	0	0	0	0	0	.58	.02	0	1.34	.29	0
22	0	.02	0	.43	0	.16	0	0	.04	0	0
23	0	.10	0	.39	0	0	0	0	0	0	0
24	0	.06	0	.07	.27	0	0	0	0	0	0
25	0	0	0	0	.45	0	0	0	.17	0	0
26	0	0	.65	0	.22	0	.18	.32	.73	0	.36
27	0	.11	0	.06	0	0	.04	.96	0	.06	0
28	0	.08	0	.17	0	0	.60	.10	0	0	0
29	0	---	0	.08	0	0	0	.53	0	0	0
30	0	---	.23	0	0	0	0	.15	2.69	0	0
31	.22	---	---	---	0	---	0	0	---	0	---
TOTAL	.39	1.18	1.09	5.49	3.87	2.89	1.09	2.16	7.32	1.62	1.31

results of the monthly data analyses are presented in graphical form in Figures VI-I through VI-4. Frequency distribution plots for the period of record are shown in Figures VI-5 through VI-8 and return period in years are shown in Figures VI-9 through VI-12. The results of the summary by storm are shown in Table VI-2. As can be seen, the average storm size is relatively small, 0.21 inches. with the average intensity being 0.04 inches/hour. Additional tabular data from the SYNOP program is included in the Data Report for those interested in further rainfall detail.

Rainfall data for the period 1957 through 1978 was also available from the Michigan State University weather station. This data was also analyzed using the SYNOP program. Table VI-2 shows the summary rainfall statistics by storm. There are differences in the average storm values between the two stations based on period of record data. Tabular and graphical results from this station, number 202395, are presented in the Data Report.

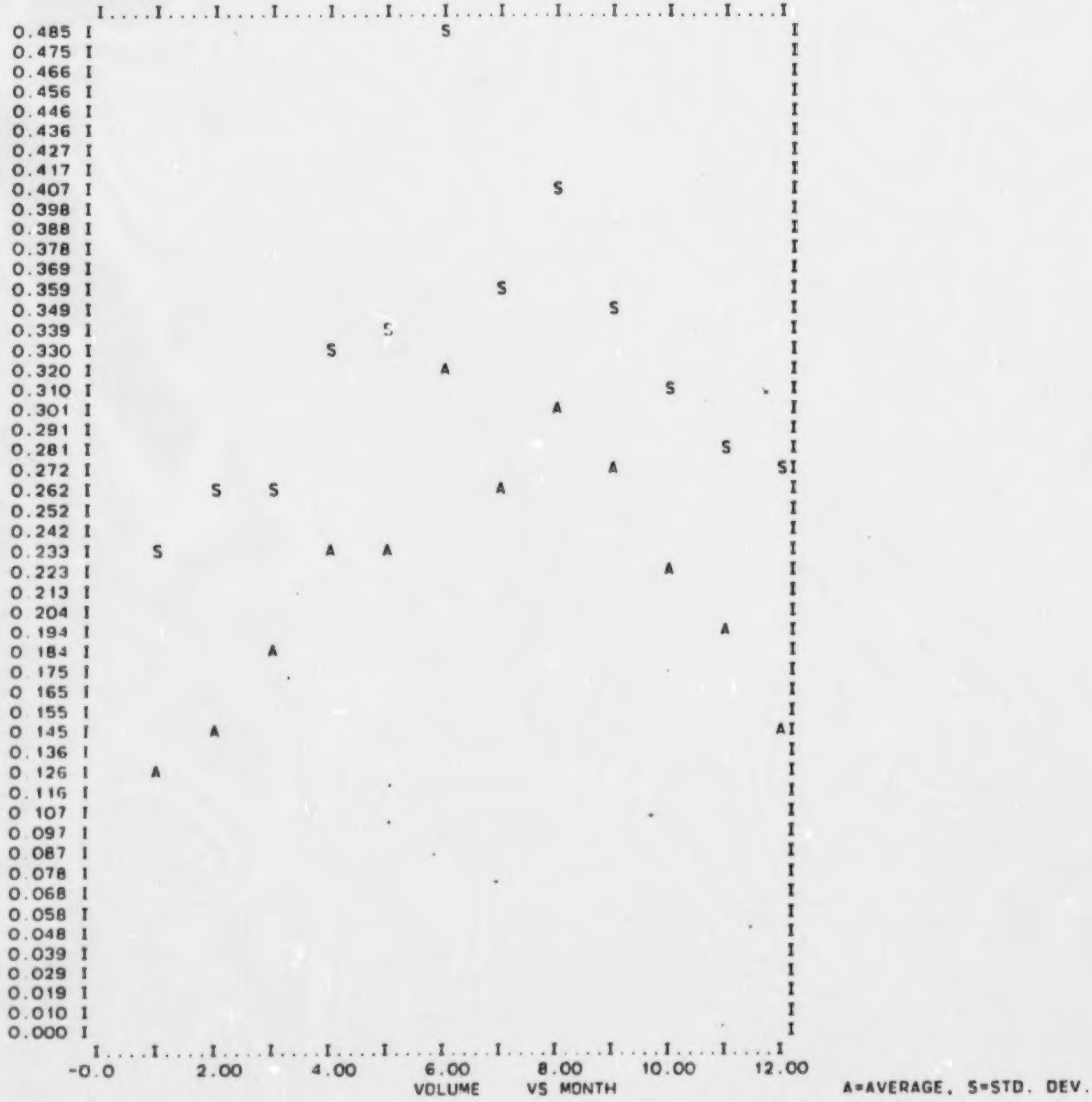

Figure VI-1 Station 204641

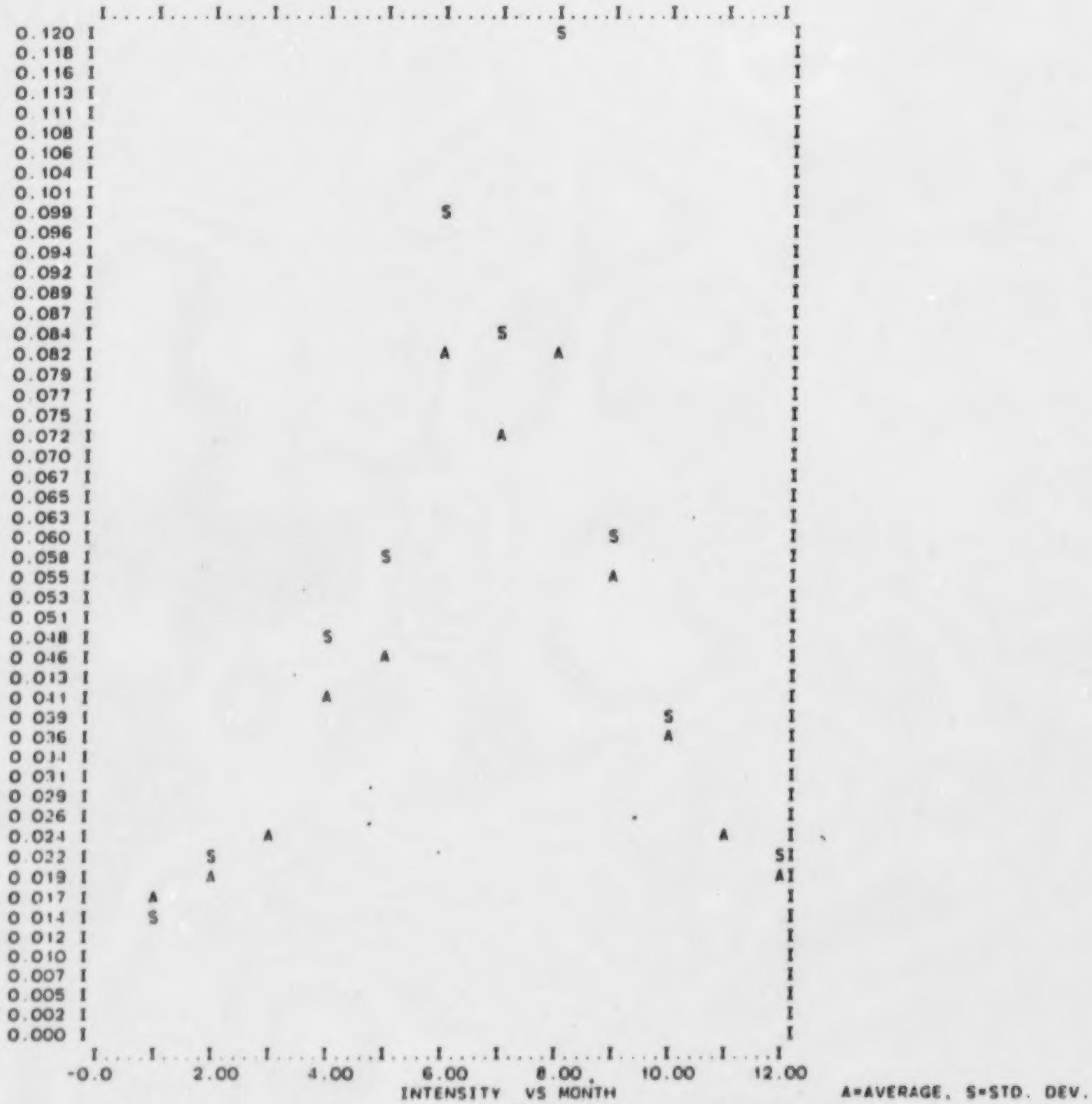

Figure VI-2 Station 204641

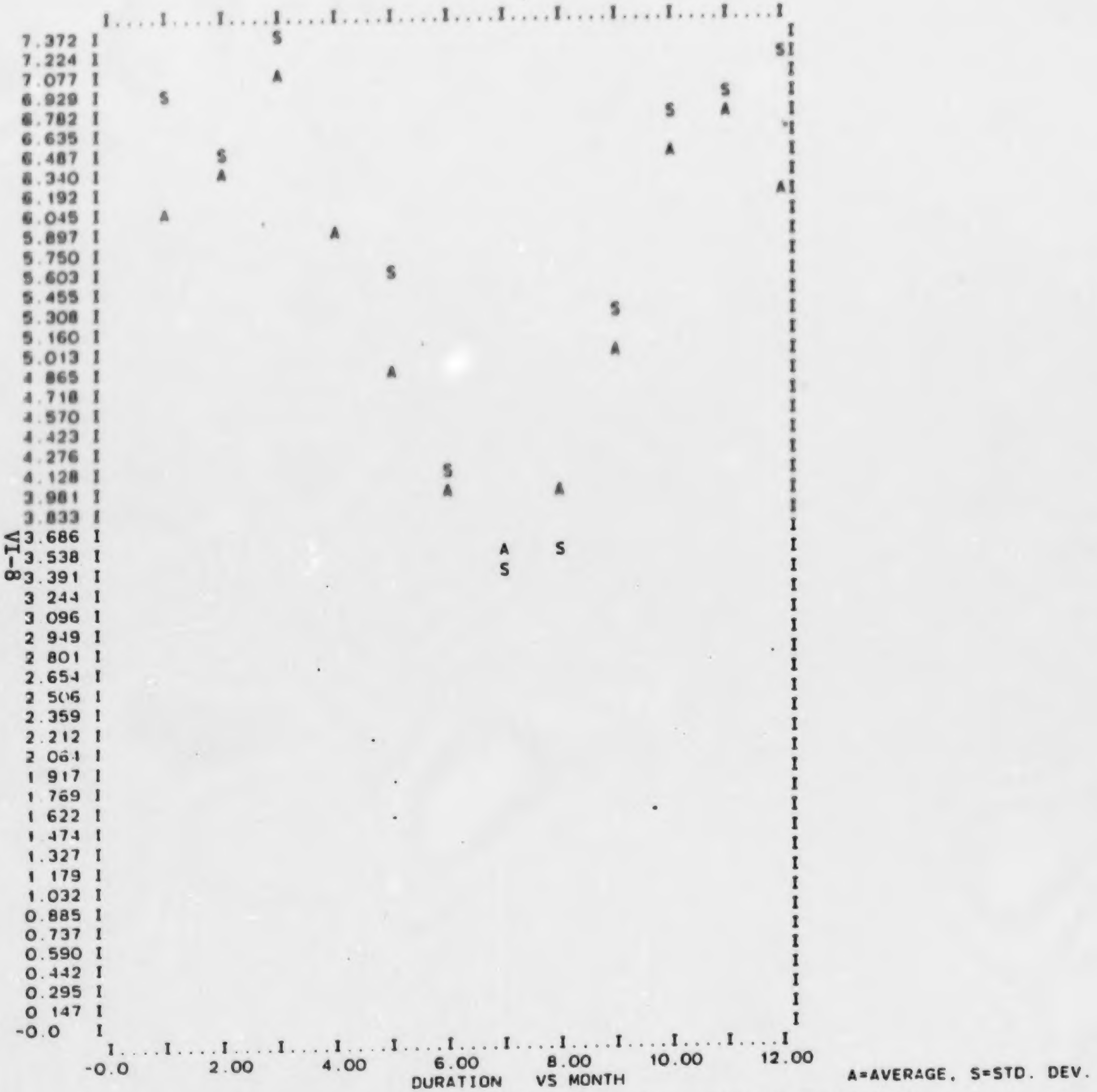

Figure VI-3 Station 204641

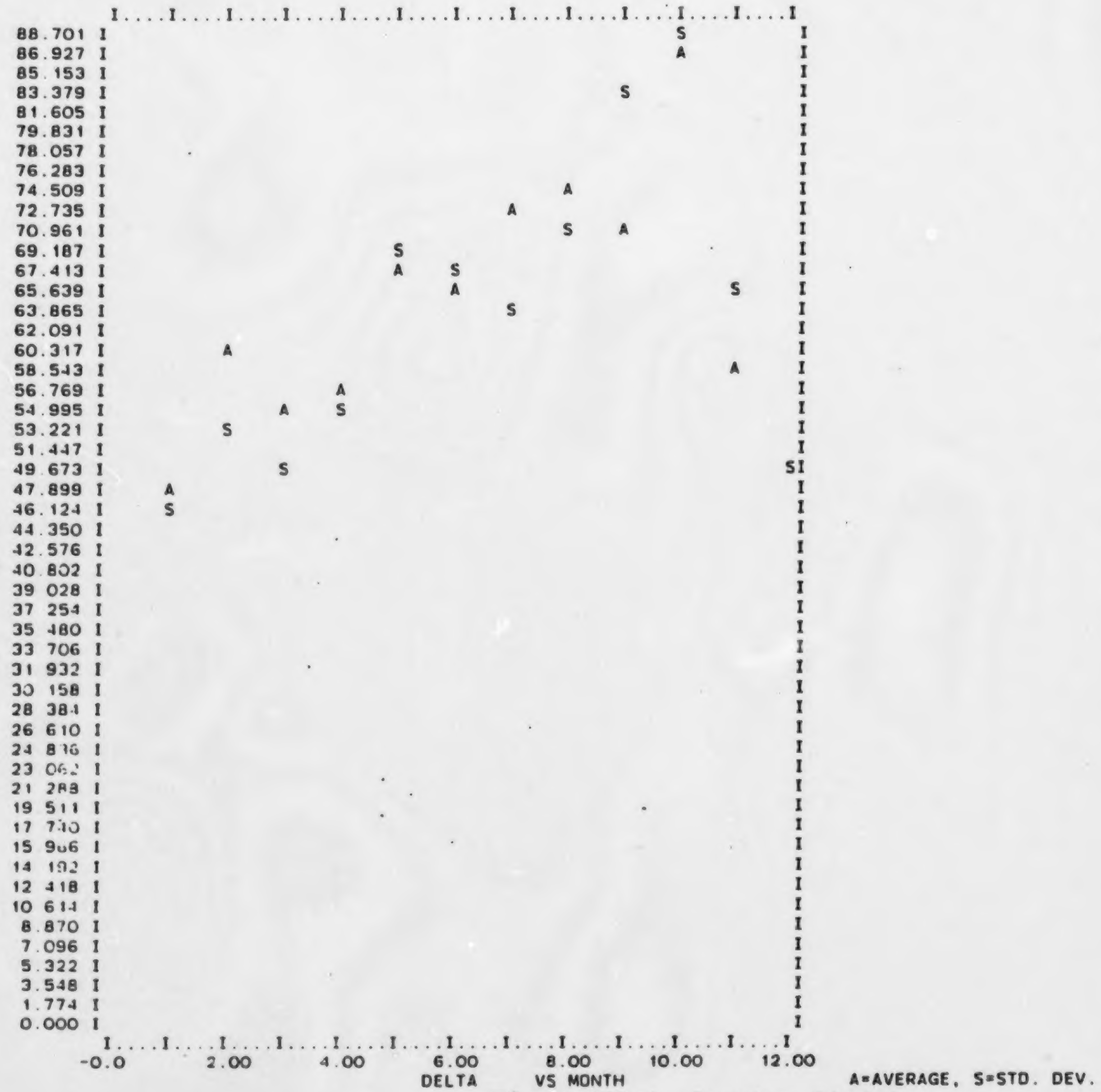

Figure VI-4 Station 204641

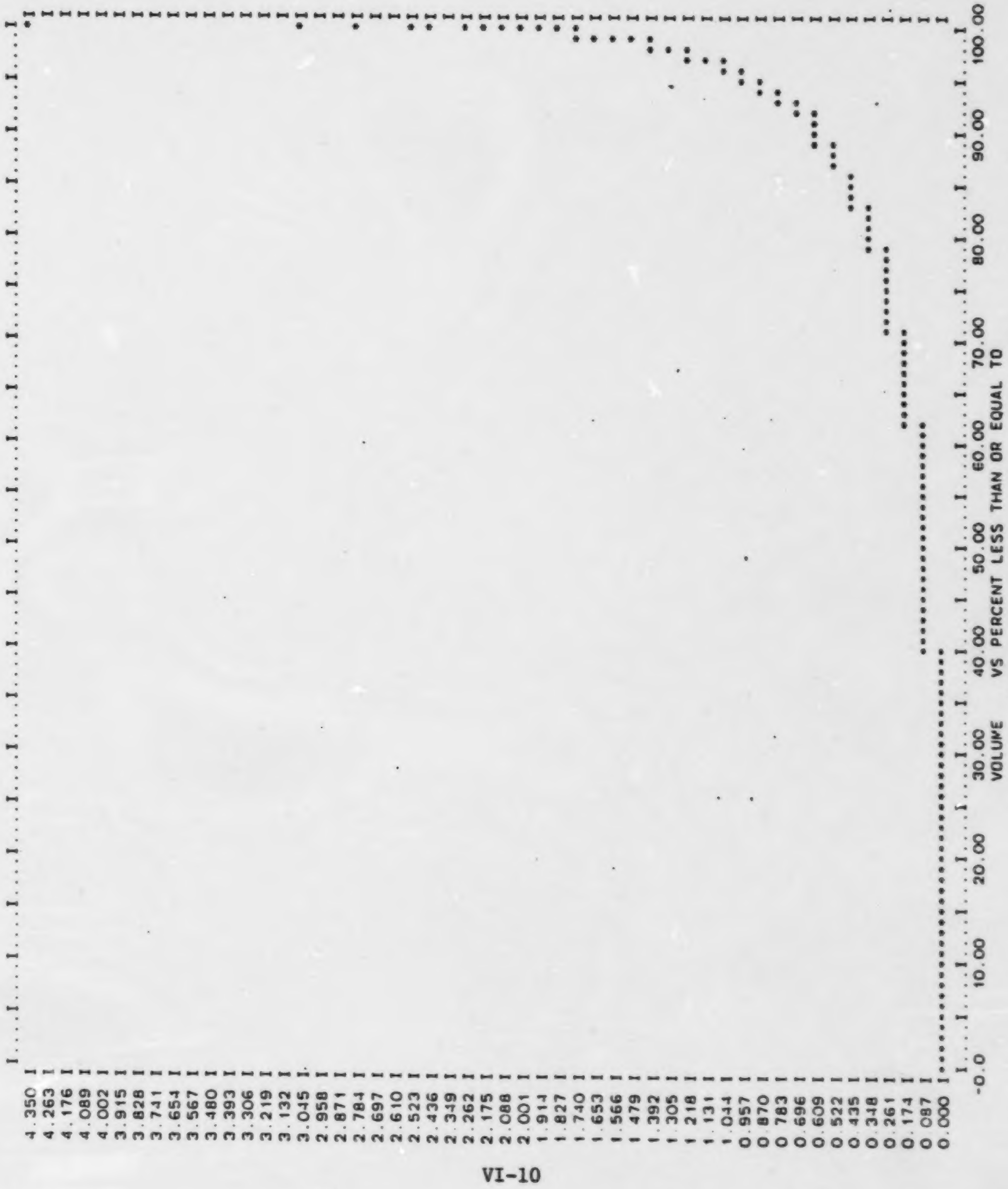

Figure VI-5 Station 204641

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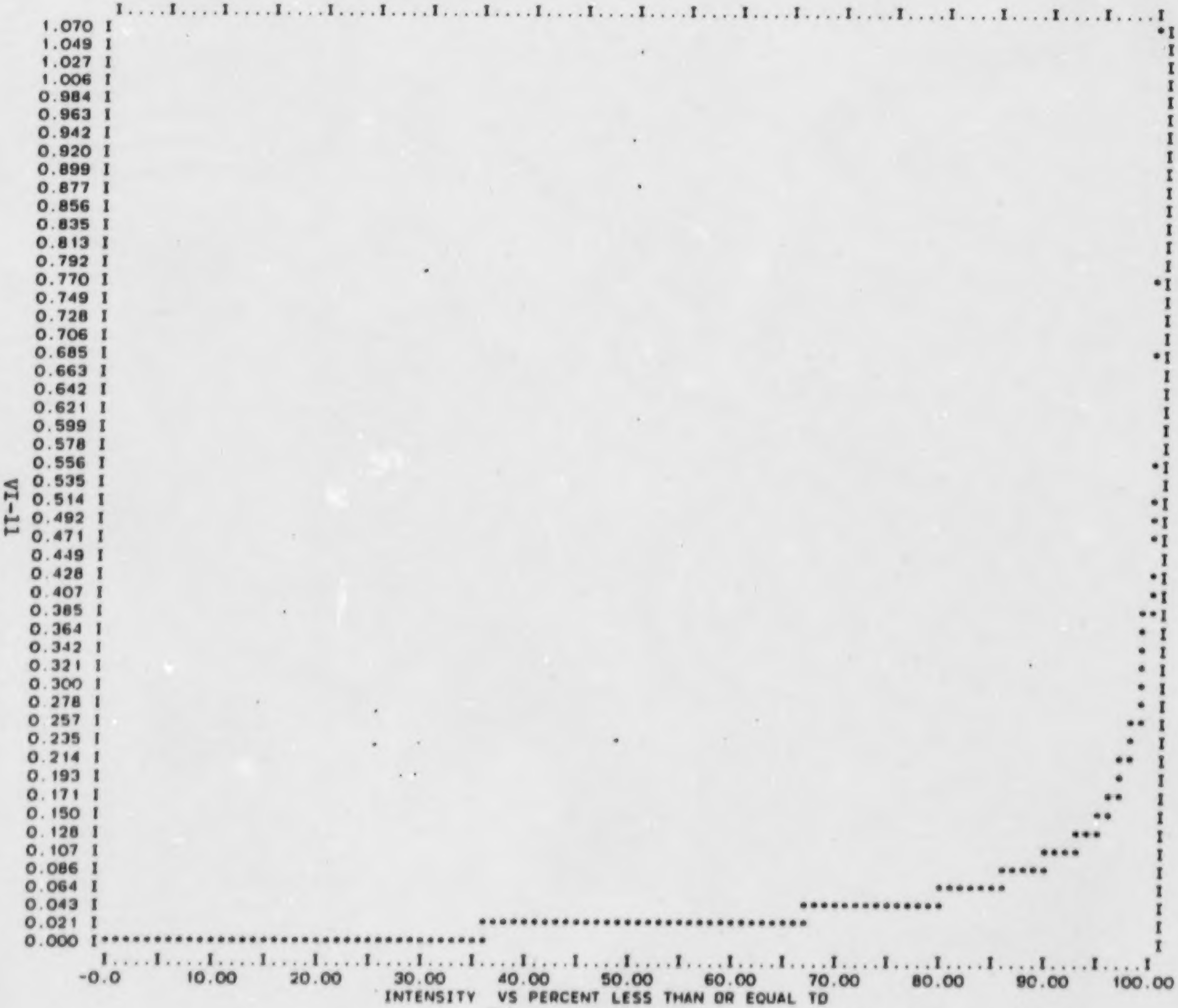

Figure VI-6 Station 204641

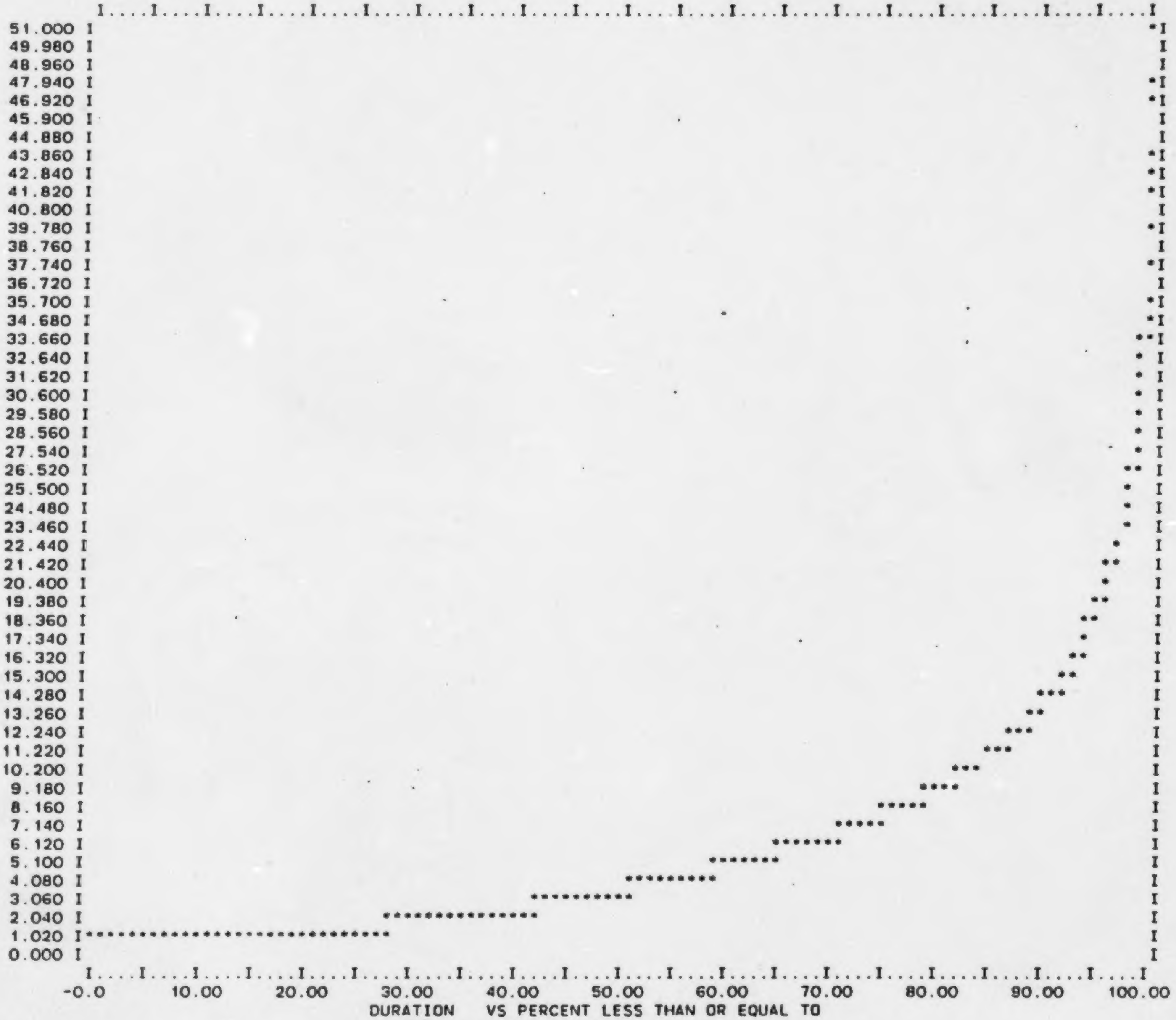

Figure VI-7 Station 204641

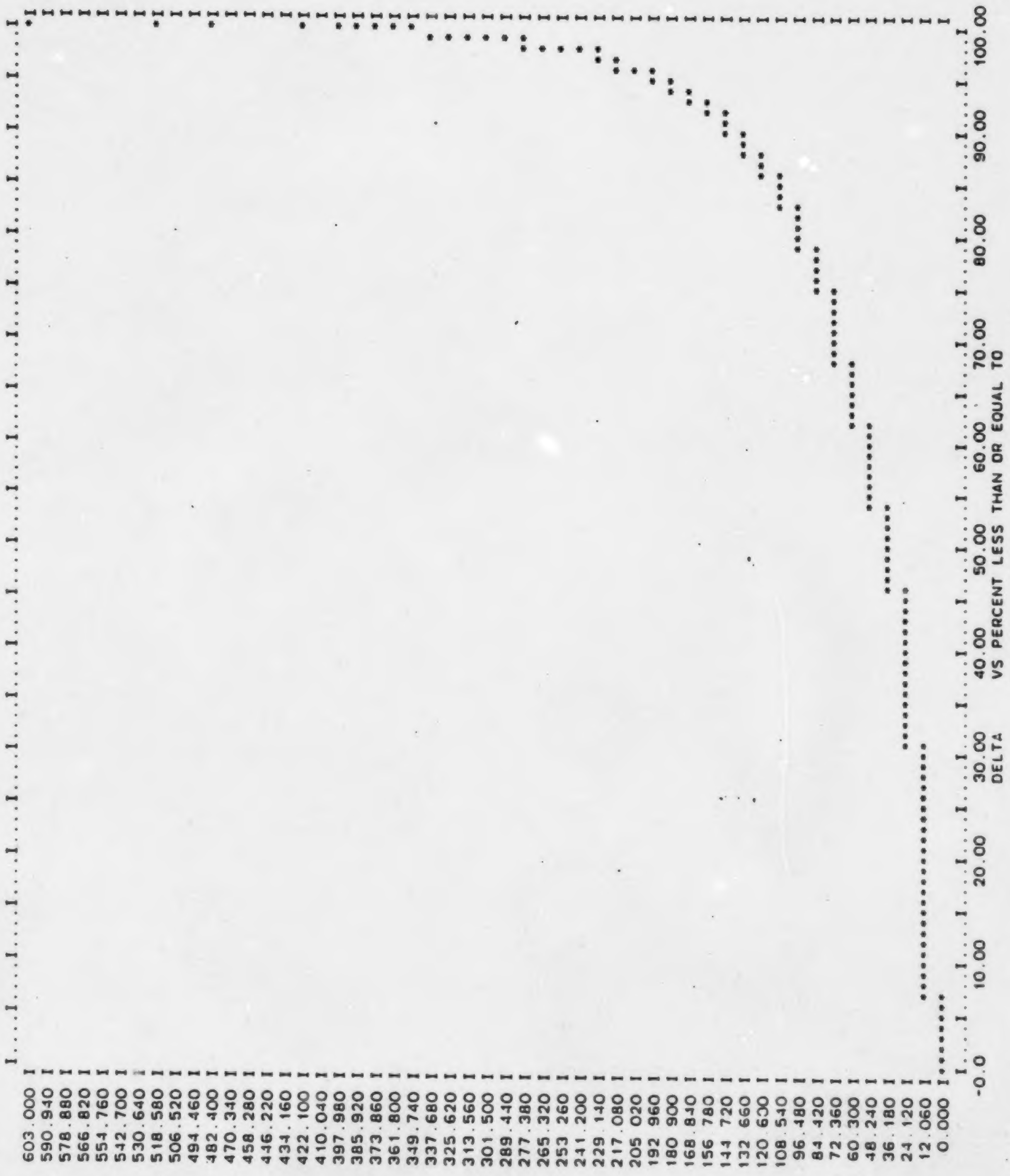

Figure VI-8 Station 204641

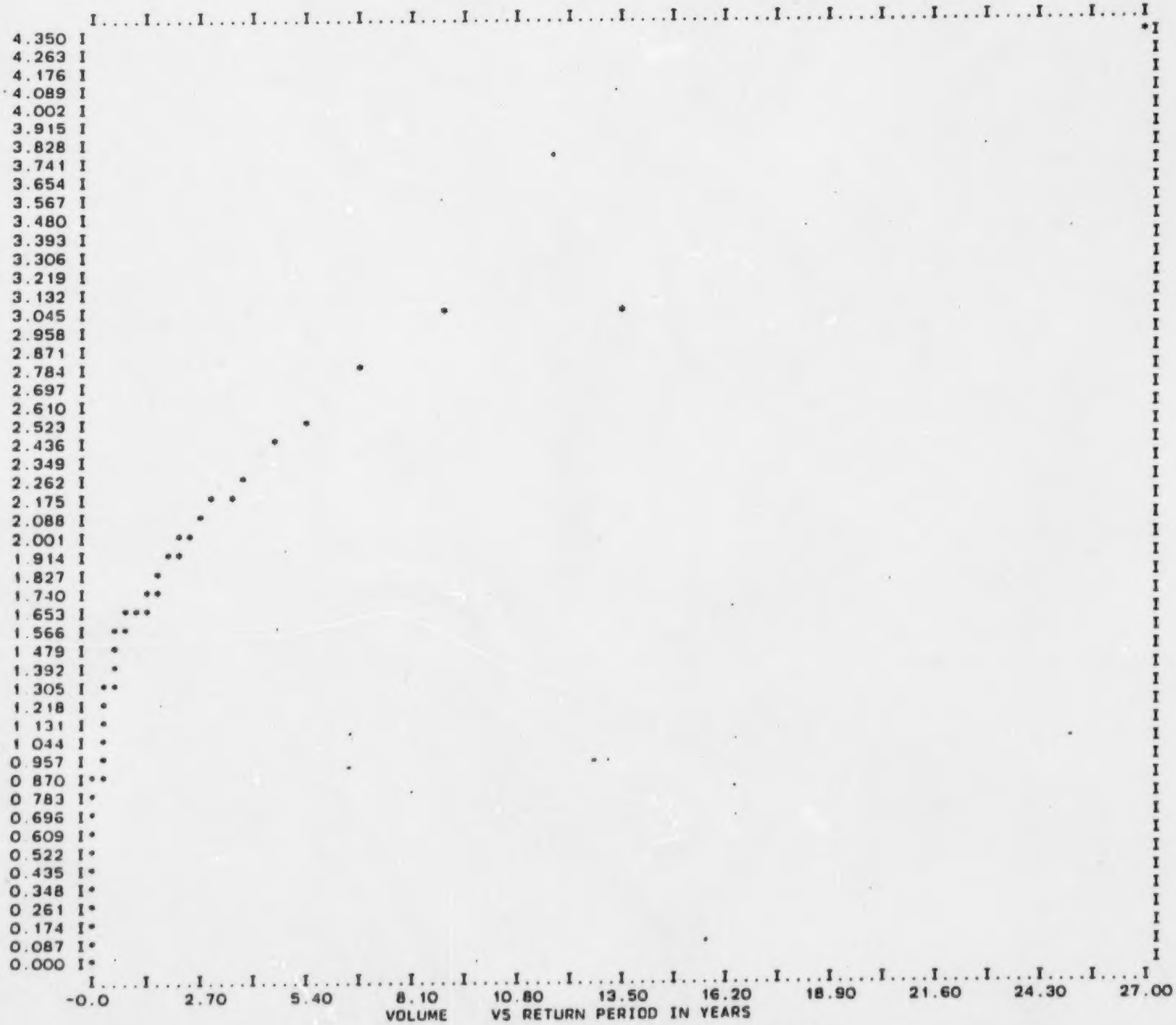

Figure VI-9 Station 204641

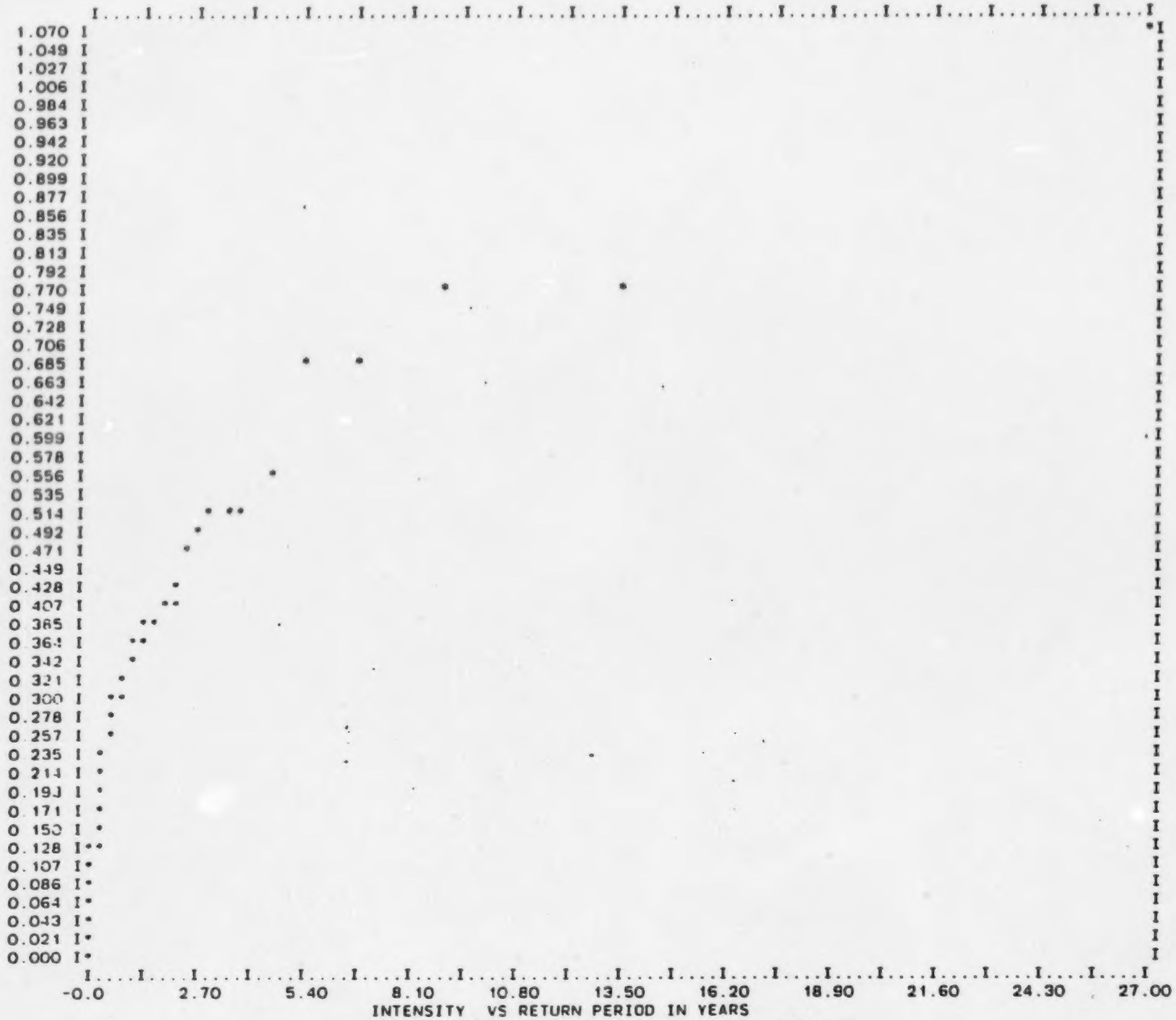

Figure VI-10 Station 204641

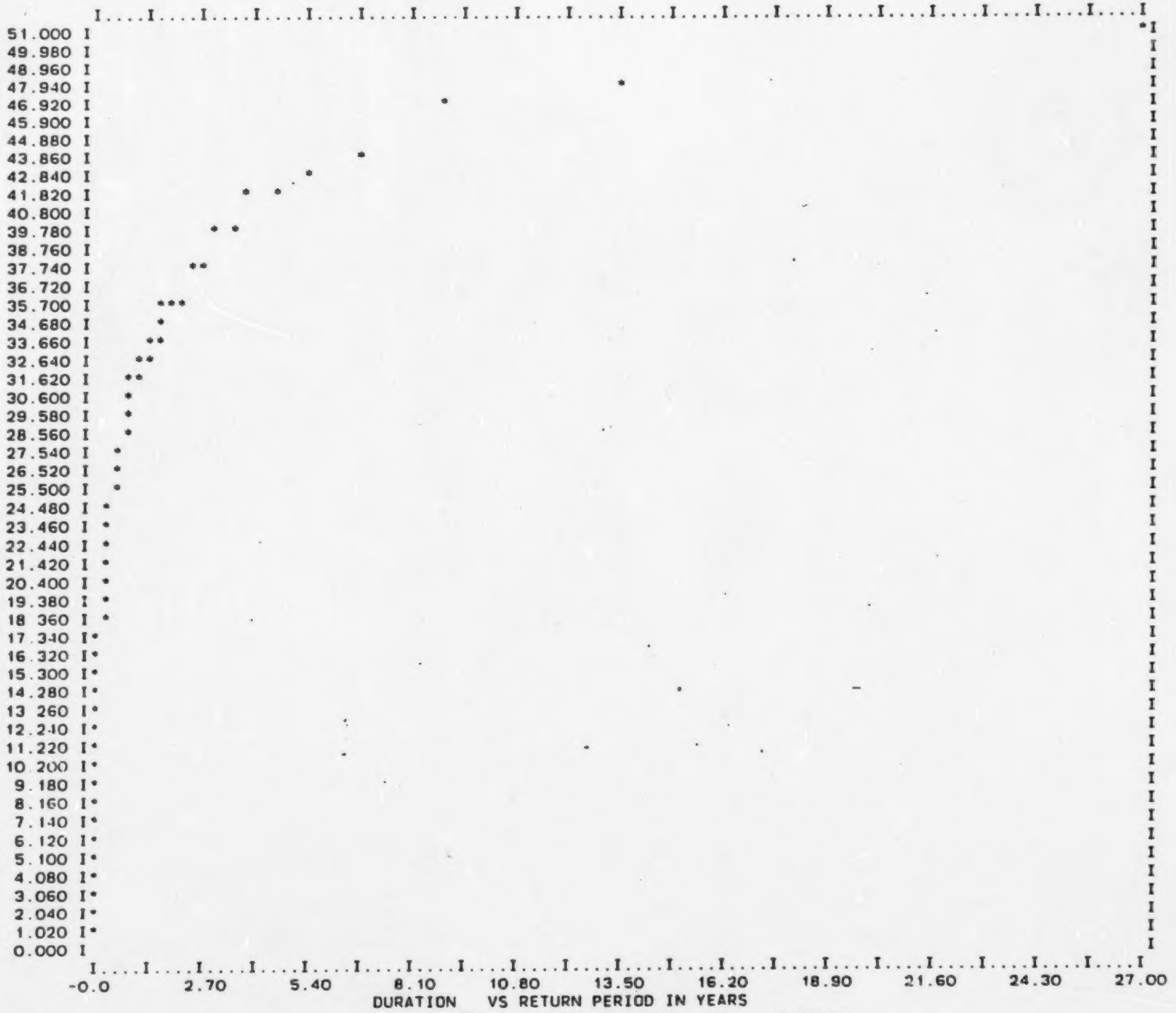

Figure VI-11 Station 204641

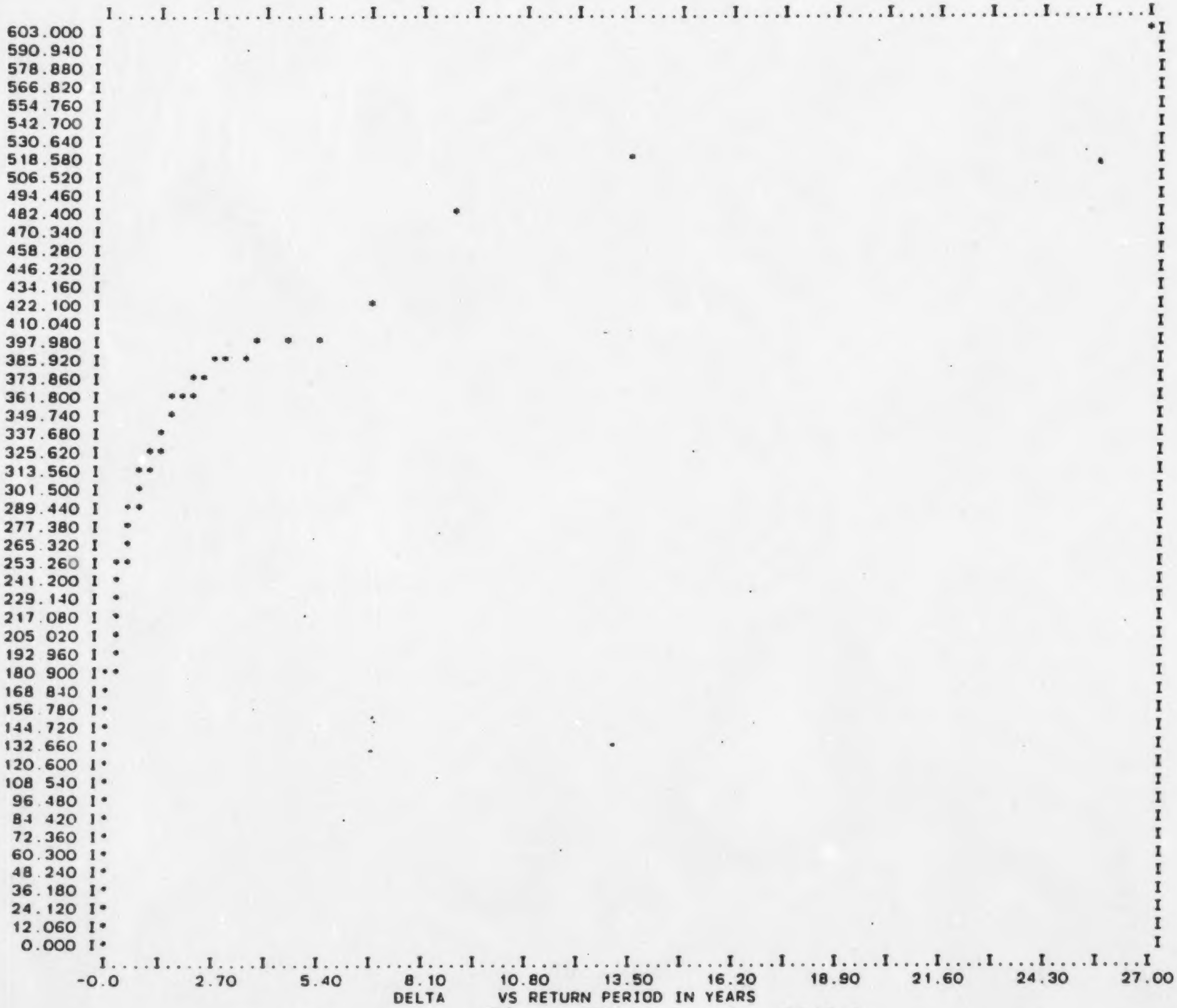

Figure VI-12 Station 204641

TABLE VII-1

Grand River at Eaton Rapids
Annual Discharge Statistics
For the Period of Record*

Mean Flow (cfs)	454
Standard Deviation	157
Coefficient of Variation	0.35

Flow Exceeded 'x' Percent of Time

x	Flow (cfs)
95	89
90	110
75	160
70	180
50	300
25	600
10	1,000

* Data for the years 1951 through 1979 were used.

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TABLE VI-2
Rainfall Statistics by Storm
Period of Record

Station 204641 (1948-78)

	Min.	Max.	Ave.	Standard Dev.	Coeff-Variation
Volume (inches)	.01	4.35	.21	.33	1.56
Intensity (inches/hr.)	.003	1.07	.04	.063	1.55
Storm Duration	1.0	51	5.6	6.16	1.10
Dry Period Between Events	6.0	603	62	63.7	1.02

Station 202395

	Min.	Max.	Ave.	Standard Dev.	Coeff-Variation
Volume (inches)	.01	4.29	.26	.37	1.42
Intensity (inches/hr.)	.003	.77	.05	.067	1.41
Storm Duration	1.0	59	6.2	5.9	0.95
Dry Period Between Events	6.0	887	87	86.8	0.995

ATMOSPHERIC DEPOSITION

Sources - Precipitation

As described earlier, rainfall intensity and volume were measured using a 7 day recording weighing bucket rain gauge. Back up data was provided by the Lansing Waste Water Treatment Plant. The treatment plant is located on the study area boundary near the outlet to the Grand River. Two manual gauges were located at the atmospheric deposition stations to check for spatial variation of rainfall volume. Rainfall data for the study period is presented in Table VI-1.

Atmospheric Load Sources

Atmospheric loading was monitored at two sites within the study area. From the beginning of the study period until August 1981, each station consisted of a bulk fallout collector, an automatic rainfall collector and a manual rain gauge. In August, 1981 one of the automatic rain collectors was converted to an automatic dryfall collector. Bulk fallout, dryfall and a rainfall composite were collected monthly. Rainfall was also collected during individual rain events.

Rainfall was collected using 11.5 inch polyethylene funnels draining into a 2 liter polyethylene bottle. Two such bottles were housed in a box connected to the automatic rain collector; the box opened when the automatic collector was tripped. Bulk fallout samples were collected in a polyethylene bucket 8.25 inches in diameter and 10 inches deep. The bucket was surrounded by a bird ring to prevent avian deposition. Dryfall was collected in a 6.25 inch diameter bucket 8 inches deep housed in an automatic collector. The automatic collector was set to open during dry periods and close when it rained.

Wetfall

Statistical analyses of individual rain events was performed. No statistically significant relationships were found between wetfall concentration and amount of rain, duration or antecedant drydays. Examination of the data indicated that concentration of selected parameters displays a log-normal distribution. Table VI-3 presents mean concentration, geometric mean and geometric deviation for the parameters of interest. A complete list of monitored parameters, on a pound per acre basis, is presented for each event in the mass load summaries in the separate Data Report.

Using the average event mean concentrations listed in Table VI-3, the wetfall load from an average size storm (0.21") and the average yearly load (27.9") are presented in Table VI-4. The rainfall amounts are those calculated by the SYNOP program as discussed earlier in this Section.

TABLE VI-3

Event Wetfall Mean Concentration

Parameter	Mean Concentration mg/l	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	6	0.763	0.474
TP	0.03	-1.501	0.371
TKN	0.74	-0.132	0.209
COD	10	1.008	0.210
Lead	0.007	-2.142	0.238
Copper	0.006	-2.246	0.398

TABLE VI-4

Wetfall Load

Parameter	Ave. Storm (lb/acre)	Ave. Year (lb/acre)
TSS	0.3	38
TP	0.001	0.2
TKN	0.04	5
COD	0.5	63
Pb	0.0003	0.04
Cu	0.00028	0.038

The amount of wetfall pollutant load entering the drainage system depends on the runoff coefficient. Tables VI-5, VI-6 and VI-7 present the range of wetfall load that can be expected to enter the drainage system using the range of observed runoff coefficients and average wetfall loads. The data are presented for stations 10, 6, and 2 which are the inlet to the upsized pipe system, inlet to the on-line retention basin, and the industrial land use station, respectively.

It is apparent from this data that the wetfall load generally makes up a small portion of the total runoff load. This would be expected in light of the relatively low runoff coefficients at these stations.

Dryfall

Dryfall was measugred using two different methods. The first method utilized an open polyethylene container that collected bulk fallout (both wet and dry). The wetfall load was then subtracted from bulk load to obtain dryfall. The second method, instituted during the final four months of the study, consisted of an automatic sampler which opened during dry weather and closed when it rained. Bulk and dryfall samples were collected on the first Tuesday of each month.

Table VI-8 presents bulk load, wetfall load, calculated dryfall and measured dryfall for the period 9/1/81 to

TABLE VI-5

Wetfall Load Contribution Station 10 Upsize Pipe System

Parameter	Range of Event Load* (lbs.)	Range of Yearly Load* (lbs.)	% of yearly Runoff Load
TSS	1-5	110-650	1-8
TP	0.003-0.02	0.6-3	3-15
TKN	0.1-0.7	15-86	20-100
COD	1.5-9	190-1,100	7-42
Lead	0.0009-0.005	0.1-0.7	2-14
Copper	0.0008-0.005	0.1-0.7	6-41

* Assume min. runoff coefficient = 0.04
max. runoff coefficient = 0.23

TABLE VI-6

Wetfall Load Contribution
Station 6 On-line Retention Basin

Parameter	Range of Event Load* (lbs.)	Range of Yearly Load* (lbs.)	% of Yearly Runoff Load
TSS	2-6	240-750	6-20
TP	0.006-0.2	1-4	10-40
TKN	0.3-0.8	30-100	30-100
COD	3-10	400-1,200	10-30
Lead	0.002-0.006	0.3-0.8	5-10
Copper	0.002-0.006	0.2-0.7	25-90

* Assume min. runoff coefficient = 0.21
max. runoff coefficient = 0.65

TABLE VI-7

Wetfall Load Contribution
Station 2 Industrial Land Use

Parameter	Range of Event Load* (lbs.)	Range of Yearly Load (lbs.)	% of Yearly Runoff Load
TSS	0.6-4	72-500	2-13
TP	0.002-0.01	0.4-3	2-12
TKN	0.08-0.5	9-66	15-100
COD	1-7	120-830	4-24
Lead	0.0006-0.004	0.08-0.5	2-10
Copper	0.0005-0.004	0.07-0.5	3-23

* Assume min. runoff coefficient = 0.03
max. runoff coefficient = 0.21

TABLE VI-8

Atmospheric Deposition 9/1/81 - 10/6/81

Parameter	Bulk (lb.acre)	Rain (lb./acre)	Calc. Dryfall. (lb./acre)	Measured Dryfall (lb./acre)
TS	21	31	-10	20
TSS	5	10	-5	4
Acidity	7	17	-10	5
TOC	3	20	-17	3
TP	0.02	0.02	0	<0.01
NH_3N	0.50	0.79	-0.29	0.02
TKN	0.7	1.2	-0.5	<0.01
NO_3+NO_2-N	0.6	1.2	-0.6	0.2
COD	7	17	-10	<5
Pb	0.01	0.02	-0.01	<0.01
Fe	0.1	0.11	-0.01	0.1
Zn	0.01	0.18	-0.17	0.04
Cr	0.01	0.02	-0.01	0.002
Cu	0.01	0.03	-0.02	<0.01
Ni	0.001	0.002	-0.001	<0.001

10/6/81. The astute observer would note that every calculated dryfall parameter for this particular one month period is a negative or zero load. This table presents an extreme example in that not every monthly composite had every parameter display a load of less than zero. However, there is a definite trend in this direction, especially with the metals and occasionally with the nutrients.

Certain problems occurred with respect to atmospheric deposition sampling. The most outstanding weakness of any program of this type is sample holding time. The collection period necessary to gather sufficient sample for analysis often exceeds the unpreserved holding time for many of the chemical species of interest. It is therefore possible some species could be degraded. For example, nitrogen species could convert to ammonia and be lost to the atmosphere; metals could adsorb on the walls of the sample vessels; etc. Hence actual pollutant loads could be underestimated.

The above problem is exacerbated when using the bulk/wetfall method. Wetfall samples are collected soon after the rain stops and therefore do not suffer any change in composition. Thus, when the dryfall calculation is made by subtracting the wetfall values from the bulk fallout, negative numbers can result.

The measured dryfall samples mentioned above were collected during dry periods only. In addition, the dryfall samples were collected in a dry medium thereby lessening the chances for aqueous reaction. It is felt that the measured dryfall samples are as representative as any sample that could be obtained under the circumstances. Table VI-9 presents dryfall as pounds per acre per day for the four months during which such samples were collected. Table VI-10 presents total dryfall load for a typical year based on the SYNOP output.

LAND USE MONITORING STATIONS

Introduction

Two stations in the drainage district were not associated with pollution control devices. Station One was located at the outlet of the study area and represented the total discharge volume and quality of the district. Station Two monitored an industrial land use area.

Station One was monitored during twenty three events over the course of the study period. Two of these twenty three events were monitored using the discrete sample approach and the remaining events were monitored by means of flow weighted composite samples. One of the discrete events was the 1981 snowmelt. A wide range of rainfall and runoff volumes were monitored with rainfall varying from 0.14 to 2.82 inches and runoff from 26,000 to 152,600 cubic feet. A summary of event characteristics is presented in Table VI-11. This table presents the range, arithmetic and geometric means and deviations for selected parameters.

TABLE VI-9

Measured Dryfall

Parameter	Units	8/4-9/1	9/1-10/6	10/6-11/3	11/3-12/1
TS	lb/acre-day	1.3	0.6	0.6	0.2
TSS	lb/acre-day	0.1	0.1	0.2	0.07
Total Acidity	lb/acre-day	0.1	0.2	0.1	0.07
Chloride	lb/acre-day	<0.04	<0.03	<0.04	<0.02
TOC	lb/acre-day	0.07	0.1	0.04	0.03
T-P	lb/acre-day	0.003	<0.0003	0.002	0.004
NH_3-N	lb/acre-day	<0.0004	0.0006	0.005	<0.0004
TKN	lb/acre-day	0.007	<0.0003	0.008	0.003
NO_3+NO_2-N	lb/acre-day	0.003	0.006	0.004	0.004
COD	lb/acre-day	<0.2	<0.2	0.2	0.2
Lead	lb/acre-day	0.0003	<0.0004	0.0004	
Iron	lb/acre-day	0.006	0.004	0.003	
Zinc	lb/acre-day	<0.0004	0.001	0.0007	
Chromium	lb/acre-day	<0.00004	0.00006	0.0003	
Copper	lb/acre-day	<0.0004	<0.0004	<0.0004	
Nickel	lb/acre-day	<0.00004	<0.00004	<0.0004	
Total Dry Days	days	27.6	33.2	27.9	27

NOTE: Less than values are a result of concentrations less than the limit of detection.

TABLE VI-10

Yearly Dryfall Load

Parameter	Ave. Load (lb/acre-day)	Yearly Load (lb/acre)
TS	0.675	220
TSS	0.12	40
Total Acidity	0.12	40
Chloride	<0.03	<10
TOC	0.06	20
T-P	0.002	0.6
NH_3-N	<0.002	<0.6
TKN	<0.004	<1.00
NO_3 + NO_3 - N	0.004	<1.0
COD	<0.2	<60
Lead	<0.0004	<0.1
Iron	0.004	1.0
Zinc	<0.0007	<0.2
Chromium	<0.0001	<0.03
Copper	<0.0004	<0.1
Nickel	<0.00004	<0.01

Total Dry Days - 321

TABLE VI-11

Summary of Event Characteristics
Station 1 Study Area Outlet

Parameter		n	Min.	Max.	Mean	Standard Deviation
Rainfall (inches)		23	0.14	2.82	0.61	0.538
Runoff Volume	(ft.^{3})	23	26,000	152,600	161,900	305,330
Runoff Coefficient		23	0.07	0.33	0.12	0.063
TSS	mg/l	23	5	395	137	104
TP	mg/l	22	0.11	0.94	0.43	0.240
TKN	mg/l	23	0.65	3.57	1.62	0.6634
BOD	mg/l	13	1	13	7	4.21
COD	mg/l	18	5	169	62	42.5
Lead	mg/l	18	0.03	0.50	0.12	0.112
Copper	mg/l	14	0.005	0.082	0.029	0.0193

TABLE VI-11 (Continued)

Summary of Event Characteristics
Station 1 Study Area Outlet

Parameter	Median Conc.	Antilog 10 Mean	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	110	98	1.9900	0.42286
TP	0.38	0.37	-0.42988	0.26523
TKN	1.54	1.5	0.17502	0.17445
BOD	7	6	0.74354	0.40310
COD	57	44	1.6447	0.44052
Lead	0.096	0.09	-1.0481	0.33121
Copper	0.020	0.024	-1.6188	0.28643

Statistical analyses were performed on the data generated at the district outlet (Station One). The results are presented in Tables VI-12. The analyses indicated a strong correlation between runoff volume and total rainfall (r=0.95) and significant correlations between loadings of the various parameters and total rainfall. As distinct from other stations monitored during the course of this study, the runoff coefficient showed a significant relationship to total rain x (r=0.65 at the 99 percent confidence level). Concentration of the various parameters were not related to total rainfall to any significant degree. Concentration of the selected parameters tended to display a log-normal distribution (Table VI-11).

Station Two was monitored during seventeen events over the course of the study period. Two events were monitored by collecting discrete samples, with the remaining events monitored composite samples. Rainfall volumes ranged from 0.14 inches to 1.05 inches, with runoff volumes ranging from 2,000 to 40,000 cubic feet. The mean runoff coefficient was 0.12 (Table VI-13), extremely low for an industrial area. However, runoff in this storm drain is diverted to an in-house treatment plant until storm generated runoff exceeds the pumping capacity of the plant. Once pumping capacity (4 cfs) is exceeded, runoff spills over a weir and into the Station Two sewer. Therefore the measured volumes of storm generated runoff are low as is the runoff coefficient. The loadings (pounds) are less than would be expected although the concentration data is representative.

TABLE VI-12

Linear Regression Statistics
Station 1 Study Area Outlet

Dependent Variable	Independent Variable	n	Correlation Coefficient	Standard Error	F Statistic	Significance
Runoff Volume	Total Rain	23	0.95	99.135	187.7	0.0000
Runoff Coefficient	Total Rain	23	0.65	0.0485	15.56	0.0007
TSS Concentration	Total Rain	23	0.05	106.7	0.053	0.8195
TSS Load	Total Rain	23	0.65	1,183	15.45	0.0008
TP Concentration	Total Rain	22	0.21	0.241	0.959	0.3390
TP Load	Total Rain	22	0.79	2.25	34.1	0.0000
TKN Concentration	Total Rain	23	0.21	0.663	1.00	0.3328
TKN Load	Total Rain	23	0.93	8.14	140.4	0.0000
BOD Concentration	Total Rain	13	0.13	4.36	0.199	0.6640
BOD Load	Total Rain	13	0.57	26.97	5.30	0.0418
COD Concentration	Total Rain	13	0.11	43.6	0.195	0.6642
COD Load	Total Rain	18	0.87	388.3	50.7	0.0000
Lead Concentration	Total Rain	18	0.17	0.114	0.409	0.4942
Lead Load	Total Rain	18	0.58	0.873	8.22	0.0112
Copper Concentration	Total Rain	14	0.23	0.019	0.686	0.4237
Copper Load	Total Rain	14	0.88	0.243	41.22	0.0000

TABLE VI-13

Summary of Event Characteristics
Station 2 Industrial Area

Parameter	n	Min.	Max.	Mean	Standard Deviation
Rainfall (inches)	17	0.14	1.05	0.58	0.267
Runoff Volume (ft.^{3})	17	2,000	40,000	15,000	12,700
Runoff Coefficient	17	0.03	0.21	0.12	0.055
TSS mg/l	17	15	240	87	59.21
TP mg/l	17	0.15	1.2	0.52	0.298
TKN mg/l	16	0.23	2.5	1.2	0.589
BOD mg/l	8	1	16	8	5.18
COD mg/l	11	20	98	62	22.63
Lead mg/l	13	0.02	0.23	0.11	0.0677
Copper mg/l	5	0.021	0.064	0.032	0.003

TABLE VI-13 (Continued)

Summary of Event Characteristics
Station 2 Industrial Area

Parameter	Median Conc.	Antilog 10 Mean	Geometric Mean (log 10)	Geometric Deviation (log 10)
TSS	75	69	1.8376	0.31699
TP	0.43	0.45	-0.34485	0.24458
TKN	1.1	1.1	0.027109	0.26045
BOD	6	6	0.75268	0.47586
COD	60	70	1.8483	0.36216
Lead	0.11	0.09	-1.0282	0.30236
Copper	0.023	0.029	-1.5370	0.2021

There was no direct way to determine the actual pumping rate in the drain during a given event over the duration of the rainfall. However the maximum pumping rate of 4 cfs which was occurring during the runoff period at the sampling station can be used to estimate the diverted flow. Using the runoff duration as monitored at Station Two and the 4 cfs rate, total runoff volume estimates were made. These values were then used to recalculate an estimated runoff coefficient. These calculated coefficients ranged from 0.2 to 0.8 with an arithmetic mean of 0.40 and a median of 0.41. These runoff coefficients are more in line with typical industrial areas and would be more representative for use in any transference or comparison of this data with other locations.

Statistical analyses were performed on the data generated at Station Two. The results are presented in Table VI-14. The linear regression of runoff volume against total rainfall indicated a fairly strong correlation (r=0.83) in spite of the diversion of runoff mentioned above. There is no statistically significant relationship between the runoff coefficient and rainfall. Linear regression analyses indicated statistically significant correlations of load to rainfall for all parameters except BOD and lead. Concentration of the various parameters, with the exception of BOD and copper, displayed no significant relationship to rainfall (Table VI-14).

Distributional analyses indicated that concentration of the various parameters is log-normal, with the exception of

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TABLE VI-14

Linear Regression Statistics
Station 2 Industrial Area

Dependent Variable	Independent Variable	n	Correlation Coefficient	Standard Error	F Statistic	Significance
Runoff Volume	Total Rain	17	0.83	7,425.8	32.36	0.0000
Runoff Coefficient	Total Rain	17	0.23	0.056	0.875	0.3643
TSS Concentration	Total Rain	17	0.06	61.0	0.054	0.8201
TSS Load	Total Rain	17	0.51	108.5	5.14	0.0385
TP Concentration	Total Rain	17	0.06	0.31	0.053	0.8217
TP Load	Total Rain	17	0.62	0.47	9.17	0.0085
TKN Concentration	Total Rain	16	0.30	0.58	1.39	0.2567
TKN Load	Total Rain	16	0.54	0.96	5.67	0.0321
BOD Concentration	Total Rain	8	0.69	4.10	5.33	0.0603
BOD Load	Total Rain	8	0.16	3.97	0.16	0.7004
COD Concentration	Total Rain	11	0.03	192	0.007	0.9370
COD Load	Total Rain	11	0.51	53.99	3.52	0.0901
Lead Concentration	Total Rain	13	0.32	0.067	1.21	0.2942
Lead Load	Total Rain	13	0.38	0.10	1.89	0.1968
Copper Concentration	Total Rain	5	0.61	0.25	1.82	0.2700
Copper Load	Total Rain	5	0.51	0.08	1.07	0.3764

COD. COD can be described as well by either a log-normal distribution or a Gaussian distribution. Therefore, for consistency, log-normal distribution is assumed for subsequent use of the data (Table VI-13).

Runoff Sources

In the investigation of the sources of runoff pollutant loads, it has been a common approach to examine the land use of various areas, and to relate these uses with levels of pollutant loads. Land use versus pollutant loading are examined herein using two different methods. In the first evaluation, land use versus event mean concentration is considered. This is followed by a consideration of land use versus load.

A geometric mean EMC has been determined based on all monitored events at each station to provide a basis for comparison between the various drainage districts. The geometric mean EMC was also used with rainfall volume and runoff coefficients to determine average event loading and annual loading for each drainage district. Inherent in the use of the median EMC for these calculations is the assumption that concentration is independent of rainfall.

The drainage areas represented by Stations Eight and Three are actually sub-districts of larger areas. The flow, load, and concentration presented in this portion of the report have been calculated by subtracting upstream values from downstream values. Table VI-15 presents the various drainage district and sub-district areas, along with the percentage of residential, parkland/institutional, commercial/industrial and impervious areas.

The drainage districts were ranked by their geometric mean EMC from highest concentration to lowest concentration in Table VI-16. Seven selected parameters were examined (TSS, TP, TKN, BOD, COD, Pb, Cu) and assigned a numerical value between one and six for each of the drainage areas. The numerical assignments were then added for each area, thereby providing a method for ranking the different land use areas on the basis of the seven key parameters. The area with the highest overall concentrations is the sub-district area above Station Eight. This district is characterized by a large amount of older residential area with predominantly ditch and swale drainage. The area has a large unpaved parking lot and school yard. Area driveways are often dirt or gravel. Visual inspection of the area also indicates some erosion in the ditch and swale drainage system. The unpaved areas and erosion very likely account for the high TSS values found at Station Eight, as well as at Station Ten, which is a similar area but without the parking lot and school area.

Normally it would be anticipated that commercial and industrial areas would display higher concentrations than a residential area. In fact, the industrial area (Station Two) occupies the second ranking and displays the highest concentration of TP, COD, and copper, however TSS and TKN

TABLE VI-15

Land Use Mix by Drainage District
(Highest - Lowest)

Station	Area (Acres)	Residential (Percent)	Park/Inst. (Percent)	Comm/Ind. (Percent)	Impervious (Percent)
#1	452.6	48	28	24	37
#2	63	0	0	100	100
#3	60.6	63	24	14	26
#6	30.3	67	0	33	68
#8	89	62	38	0	17
#10	74.9	48	30	22	38

TABLE VI-16

Event Mean Concentration by Rank
(Highest - Lowest)

Station	TSS (mg/l)	TP (mg/l)	TKN (mg/l)	BOD (mg/l)	COD (mg/l)	Lead (mg/l)	Copper (mg/l)
#8	145	0.42	2.4	7	61	0.09	0.01
#2	69	0.45	1.1	6	70	0.09	0.029
#1	98	0.37	1.5	6	44	0.09	0.02
#10	120	0.32	1.4	5	53	0.08	0.02
#3	75	0.31	1.9	4	45	0.10	0.005
#6	52	0.12	1.3	7	49	0.08	0.01

values are lower. Station Six, which possesses a large amount of commercial area (Shopping Center) has the lowest overall concentration ranking. The low concentration values may be somewhat unexpected considering the relatively large amount of commercial and high density residential uses in this area. The low concentrations may be a result of a dilution effect due to the large amounts of runoff generated by the area. The runoff coefficient at Station Six is approximately 3 times that of the other stations (Table VI-17).

Examination of event mean concentration provides a comparison ot the water quality of the runoff from the various land mix types. However, high concentrations are not necessarily critical in terms of effects on the river if the runoff volumes associated with the high concentration, are small. Runoff volume is dependent on rainfall amounts and the runoff coefficient for the drainage area. The runoff coefficient is generally dependent on such factors as land use, imperviousness, and rainfall intensities. Highly urbanized areas generally have higher runoff coefficients than less urbanized areas. Therefore, highly urbanized areas would be expected to deliver larger loads on a pound per acre basis, given comparable event mean concentrations.

Table VI-17 presents the annual pollutant load from each draingae district. The districts are ranked by a numerical value associated with each parameter load in the same manner as the EMC ranking. Table VI-18 presents the annual pollutant load on a pounds per acre basis. The data

TABLE VI-17

Annual Pollutant Load by Basin
(Highest - Lowest)

Pounds/Year

Station	Runoff Coefficient	TSS	TP	TKN	BOD	COD	Lead	Copper
#1	0.12	34,900	134	534	2,100	15,700	32	7
#8	0.11	9,300	27	154	450	3,930	6	0.6
#6	0.38	3,900	9	98	525	3,675	6	0.8
#2	0.12	3,400	22	55	298	3,480	5	1.4
(#2)*	(0.4)	(11,000)	(73)	(180)	(990)	(11,600)	(17)	(4.7)
#10	0.12	7,130	19	83	297	3,150	5	1.2
#3	0.14	4,170	17	106	222	2,500	6	0.3

* Estimated using calculated runoff coefficient.

is ranked using the same method as the EMC ranking. The loads for both Table VI-17 and VI-18 were calculated using average annual rainfall, mean runoff coefficient and geometric mean EMC for each station. In addition, an estimated load for Station Two is presented. The estimated load was calculated by estimating the actual runoff coefficient including the flow diverted by the pumping station in this drain as discussed above.

Station Six displays the highest loading on a pound per acre basis (Table VI-18). It occupies this position due to the large amount of runoff generated in the area. The other areas, which have similar runoff coefficients, maintain the same relative rankings displayed in the Event Mean Concentration ranks (Table VI-16). The Station Two estimated load, (in parenthesis in Tables VI-17 and VI-18) would have the highest overall ranking. The ranking would change due to the increased volume of runoff and hence increased load. These results show that loads generated in the Bogus Swamp Drain district are primarily a result of differences in flow. The concentration values are all fairly close, with some exceptions, and indicate there is not a great deal of difference in runoff water quality among the various land use mixes.

On a load basis the land use type exhibiting the heaviest pollution is the industrial (assuming no diversion of runoff), followed by commercial/residential, and residential areas.

TABLE VI-18

Annual Pollutant Load Per Acre
(Highest - Lowest)

Pound/Acre Per Year

Station	Runoff Coefficient	TSS	TP	TKN	BOD	COD	Lead	Copper
#6	0.38	129	0.3	3.2	17	121	0.2	0.02
#8	0.11	105	0.3	1.8	1.5	44	0.06	0.007
#2	0.12	54	0.4	0.9	4.7	55	0.07	0.02
(#2)*	(0.4)	(180)	(1.3)	(3.0)	(16)	(180)	(0.2)	(0.07)
#1	0.12	77	0.3	1.2	4.7	35	0.07	0.02
#10	0.12	95	0.3	1.1	4	42	0.06	0.02
#3	0.14	69	0.3	1.7	3.7	41	0.09	0.005

COMPARISON OF SOURCE LOADS

The 208 study performed by the Tri-County Regional Planning Commission (TCRPC) included a data collection program to characterize urban runoff in the Lansing, Michigan area. Seven test areas were monitored including two low density residential areas, two high density residential areas, two commercial areas and an industrial area. Table VI-19 summarizes the findings of the earlier 208 work, and also presents a summary of all of the NURP project data through September. 1981.

Comparing the results of the current study, shown in Table VI-16, with the other results presented in Table VI-19 shows that generally all of the concentration values are similar. The one noticeable difference between the current study and the earlier TCRPC work is in the BOD data. The results from the Bogus Swamp Drain District are significantly lower than the earlier 208 results. Suspended solids data for the various studies are relatively close with the data collected in this study tending to be on the higher side of the earlier TCRPC data. Comparison of the nutrient data shows very close agreement for the data sets with all results being within the expected range of values.

The fact that the data collected here is in the same range as the earlier collected information further supports the loading information generated as part of the Water Quality Management Plan for the Tri-County Region. The earlier work showed only minor variations in the concentrations of various pollutants among the urban land

TABLE VI-19

Comparison of TCRPC, ENCOTEC and NURP Data

	TSS (mg/l)	TP (mg/l)	TKN (mg/l)	BOD (mg/l)	COD (mg/l)	Pb (mg/l)
TCRPC LDR	64	0.32	1.2	27		
TCRPC MDR/HDR	36	0.50	1.4	22		
TCRPC Other Urban	68	0.28	1.1	28		
NURP	77	0.24			59	.15

use types. These variations are generally within the range of levels typically encountered in stormwater runoff. Further, the significnat difference in pollutant loads (pounds/acre) between commercial/industrial and residential areas was found to be primarily due to the increased volume of runoff associated with the commercial/industrial areas. Recognition of this factor is important when considering the type, and particularly the size, of various control alternatives.

SECTION VII

RECEIVING WATER EVALUATION

INTRODUCTION

Water quality information for the Grand and Red Cedar Rivers in the vicinity of the Lansing and East Lansing urban area, and the impacts of urban stormwater runoff on these waters. are presented in this section. The character and conditions of the rivers are described through river hydrology, beneficial uses, waste source inputs, and historical water quality data. The impacts of urban stormwater runoff will be determined by integrating the "Mass Load Data" (pollutant loads) collected from the NURP monitoring stations in the Bogus Swamp Drain Drainage District with land use measurements and projections and possible pollution mitigation measures.

BENEFICIAL USES AND WATER QUALITY STANDARDS

The Red Cedar and Grand Rivers are designated for protection, as are all waters of the state, for the following: agricultural use, navigation, industrial water supply, public water supply at the point of water intake, warm water fish, and partial body contact recreation. In addition, the Moores Park Impoundment on the Grand River (at approximate river miles 157.6 -155) is designated for protection for total body contact recreation. Hydroelectric power generating facilities exist at two locations on the Grand River in Lansing. Both the Grand and Red Cedar Rivers are important to the area for aesthetics and a wide variety of recreational activities. There are several riverside parks, canoe liveries, and a zoo in the Lansing/East Lansing urban area.

The purpose of the Michigan water quality standards, as stated in the regulations, is as follows:

> Rule 1041. It is the purpose of the water quality standards as prescribed by these rules to establish water quality requirements applicable to the Great Lakes, their connecting waterways and all other surface waters of the state, which shall protect the public health and welfare, enhance and maintain the quality of water, serve the purposes of United States Public Law 92-500 and the commission act; and which shall protect the quality of waters for recreational purposes, public and industrial water supplies, agricultural uses, navigation and propagation of fish, other aquatic life and wildlife.

"Agricultural water use" means a use of water for agricultural purposes, including but not limited to livestock watering, irrigation, and crop spraying. "Industrial water supply" means a water source not protected for public water supply and intended for use in commercial or industrial applications and non-contact food processing.

"Public water supply" means a surface raw water source which, after conventional treatment will provide a safe, clean, potable and aesthetically pleasing water for uses which include but are not limited to human consumption, food processing and cooking and as a liquid ingredient in foods and beverages. "Total body contact recreation" means an activity where the human body may come into direct contact with water to the point of complete submergence, including but not limited to activities such as swimming, water skiing and skin diving. The preceding definitions were taken from the Michigan Water Quality Standards. a copy of these standards is presented in Appendix II, the standards for selected parameters of interest are presented below.

A. Suspended Solids

There is no numerical standard. Rule 1050 states: "All waters of the state shall contain no unnatural turbidity, color, oil films, floating solids, foams, settleable solids or deposits in quantities which are or may become injurious to any designated use."

B. Dissolved Solids

Rule 1051. (1) The addition of any dissolved solids shall not exceed concentrations which are or may become injurious to any designated use. Point sources containing dissolved solids shall be considered by the commission on a case-by-case basis and increases of dissolved solids in the waters of the state shall be limited through the application of best practicable control technology currently available as prescribed by the administrator of the United States Environmental Protection Agency pursuant to Section 304 (b) of United States Public Law 92-500, except that in no instance shall total dissolved solids in the waters of the state exceed a concentration of 500 milligrams per liter as a monthly average nor more than 750 milligrams per liter at any time, as a result of controllable point sources.

C. Hydrogen ion concentration

Rule 1053. The hydrogen ion concentration expressed as pH shall be maintained within the range of 6.5 to 8.8 in all waters of the state except as otherwise prescribed by Rule 1080. Any artificially induced variation in the natural pH shall remain within this range and shall not exceed 0.5 units of pH.

D. Plant nutrients

Rule 1060. Nutrients originating from domestic, industrial, municipal or domestic animal sources shall be limited to the extent necessary to prevent stimulation of growths of aquatic rooted, attached and floating plants, fungi or bacteria which are or may become injurious to the designated uses of the waters of the state. Phosphorus which is or may readily become available as a plant nutrient shall be controlled from point source discharges by the application of methods utilizing best practicable waste treatment technology for control of total phosphorus, with the goal of achieving a monthly average effluent concentration of one milligram per liter as P.

E. Fecal Coliform

Rule 1062. (1) Waters of the state protected for total body contact recreation shall contain not more than 200 fecal coliforms per 100 milliliters; and all other waters of the state shall contain not more than 1,000 fecal coliforms per 100 milliliters. These concentrations may be exceeded if due to uncontrollable non-point sources.

(2) Compliance with the fecal coliform standards prescribed by subrule (1) shall be determined on the basis of the geometric average of any series of 5 or more consecutive samples taken over not more than a 30-day period.

F. Dissolved oxygen; Great Lakes, connecting waterways and inland streams

Rule 1064. A minimum of 6 milligrams per liter of dissolved oxygen in all Great Lakes and connecting waterways shall be maintained and, except for inland lakes as prescribed in Rule 1065, a minimum of 6 milligrams per liter of dissolved oxygen shall be maintained at all times in all inland streams designated by these rules to be protected for coldwater fish. In all other waters, except for inland lakes as prescribed by Rule 1065, a minimum of 5 milligrams per liter of dissolved oxygen shall be maintained as a daily average and no single value shall be less than 4 milligrams per liter in waters naturally capable of supporting warm water fish.

Changes to the current (1973) water quality standards were proposed in January, 1982. The most significant of these changes applies to Rule 1057. This rule, if adopted, would affect waters of the state protected for public water supply sources. It would prohibit exceeding the maximum contaminant levels for inorganic and organic chemicals specified in the National Interim Primary Drinking Water Regulations, 40 C.F.R., Section 141.11 and 141.12. The waters of the Grand and Red Cedar Rivers in the Lansing area are not used as public water supply sources, so this proposed rule change would have little, if any, impact in the study area. The proposed rule is attached to the State Water Quality Standards in the Appendix.

As can be seen in the above water quality standards, a problem exists in that there are no numerical standards for some very important parameters (i.e. total suspended solids, phosphorus, nitrogen). The problem is further compounded by exceptions to the numerical standards for "uncontrollable" non-point sources as in the case for fecal coliform. Basically there is no comprehensive list of numerical standards to compare with storm water qulaity data. However, in addition to the established state standards, a set of guidelines or targeted goals have been defined by several governmental agencies for various chemical constituents. These guidelines have been compiled and are attached to the State Water Quality Standards in the Appendix. Key parameters included in these guidelines are:

Suspended Solids	25	mg/l
Total Phosphorus	<0.5	mg/l
Ammonia Nitrogen	0.4	mg/l
Total Organic Nitrogen	<0.8	mg/l

HYDROLOGIC AND PHYSICAL CHARACTERISTICS

The Grand River Basin, which drains into Lake Michigan, consists of approximately 5,600 square miles of the southwestern portion of lower Michigan. The Grand River and its tributaries drain portions of Washtenaw, Jackson, Livingston, Ingham, Eaton, Barry, Shiawassee, Clinton, Gratiot, Ionia, Montcalm, Kent, Newaygo, Ottawa, and Muskegon Counties. The major cities in the basin include Jackson, Lansing, and Grand Rapids. The Grand River Basin is approximately 125 miles long, and the width is approximately 80 miles at the widest point.

The Grand River flows north from its headwaters in Jackson County, through Eaton County and into Lansing in Ingham County where it is joined by the Red Cedar River. The Red Cedar River begins in Cedar Lake in Livingston County and flows in a westerly direction along a course of roughly 50 miles to its confluence with the Grand River. The total drainage area of the Grand River at Lansing is about 1,230 square miles, with approximately 355 square miles of this total area being contributed by the Red Cedar. The upstream boundaries of the Lansing/East Lansing urban area are assumed for evaluation purposes to be Creyts Road on the Grand River (mile 162 from the river mouth) and Okemos Road on the Red Ceder River (mile 8.5 from the river mouth). The downstream boundary is assumed to be Webster Road (mile 146 from the river mouth). This area includes Lansing, East Lansing, Lansing Township, Delta Township, and Okemos.

Hydrologic statistics were obtained from the U.S. Geological Survey for two gauging stations on the Grand River, and one gauging station on the Red Cedar River. The Grand River Basin and the locations of the U.S.G.S. gauging stations in the basin are shown in Figure VII-1.

U.S.G.S. gauging station number 04113000, Grand River at Lansing, is located in the middle of the city of Lansing. It is thirty feet upstream from the bridge on North Grand River Avenue. This is two miles downstream from the confluence of the Grand and Red Cedar Rivers, and at mile 152 from the mouth of the Grand River. The period of record is March, 1901 to September, 1906 and October 1934 to the current year. Monthly discharge statistics only are available for some periods. The records are considered good by the U.S.G.S., although they report large diurnal fluctuations at medium and low flows due to power plants above the station. The average discharge at this station is 826 cfs. The maximum discharge for the period of record, 24,500 cfs, occurred on March 26, 1904, and the minimum flow for the period of record, 2.8 cfs, occurred on September 9, 1963.[5,6]

Gauging station number 04112500, Red Cedar River at East Lansing, is located on the campus of Michigan State University in East Lansing. The gauge is in the left downstream bridge abutment of the Farm Lane Bridge. This is 5.6 miles upstream from the mouth of the Red Cedar. The period of record is from August, 1902 to December, 1903 and March, 1931 to the current year. Monthly discharge

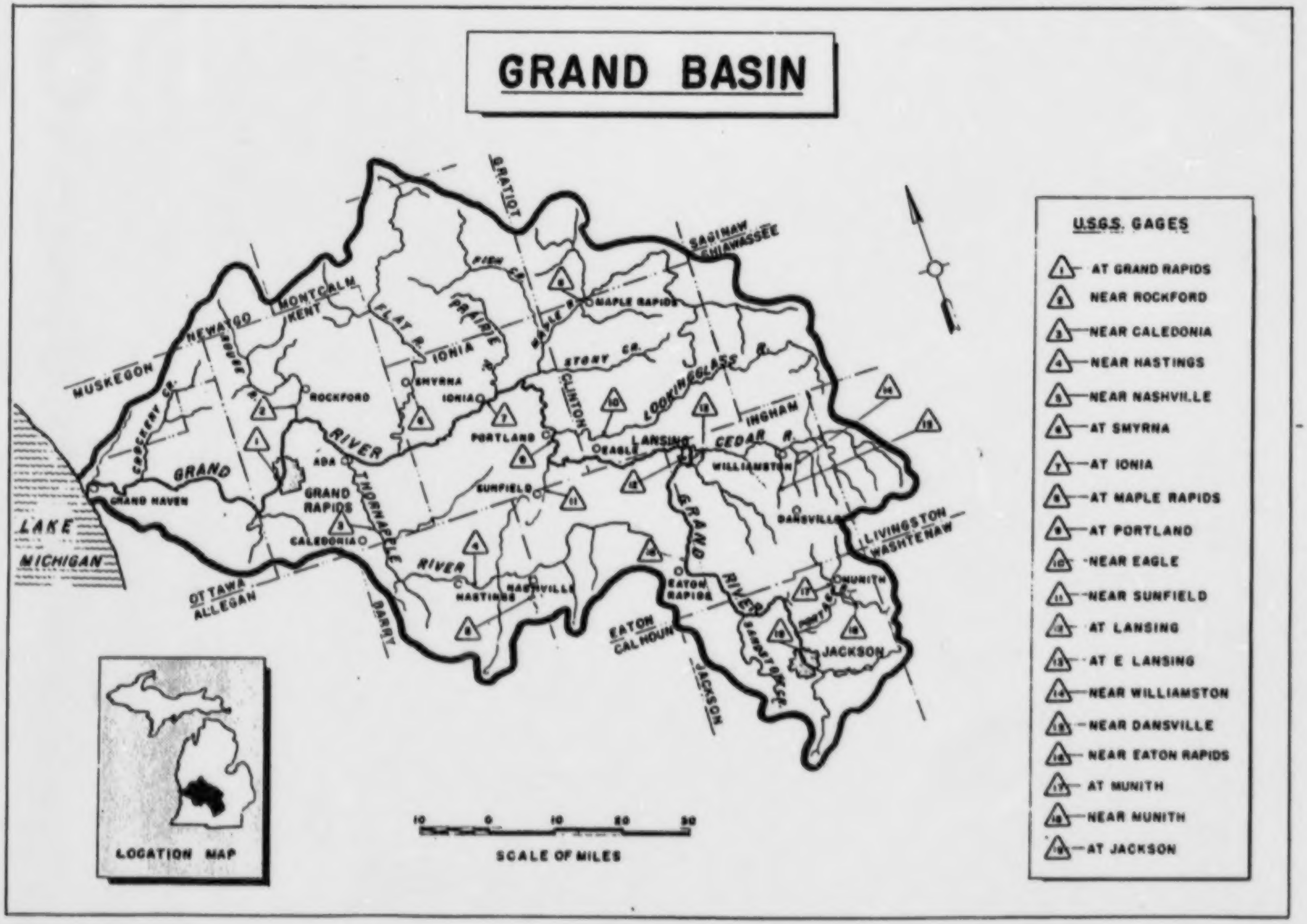
GRAND BASIN
U.S.G.S. GAGES
1 AT GRAND RAPIDS
2 NEAR ROCKFORD
3 NEAR CALEDONIA
4 NEAR HASTINGS
5 NEAR NASHVILLE
6 AT SMYRNA
7 AT IONIA
8 AT MAPLE RAPIDS
9 AT PORTLAND
10 NEAR EAGLE
11 NEAR SUNFIELD
12 AT LANSING
13 AT E LANSING
14 NEAR WILLIAMSTON
15 NEAR DANSVILLE
16 NEAR EATON RAPIDS
17 AT MUNITH
18 NEAR MUNITH
19 AT JACKSON
LAKE MICHIGAN
GRAND HAVEN
GRAND
GRAND RAPIDS
ADA
CALEDONIA
ROCKFORD
SMYRNA
IONIA
PORTLAND
SUNFIELD
HASTINGS
NASHVILLE
EAGLE
LANSING
WILLIAMSTON
DANSVILLE
EATON RAPIDS
MUNITH
JACKSON
MAPLE RAPIDS
RIVER
THORNAPPLE
FLAT R.
PRAIRIE
IONIA R.
FISH CR.
MAPLE R.
STONY CR.
LOOKINGGLASS R.
CEDAR R.
GRAND RIVER
SANDSTONE CR.
PORTAGE
CROCKERY CR.
ROGUE R.
MUSKEGON
NEWAYGO
MONTCALM
KENT
GRATIOT
CLINTON
SAGINAW
SHIAWASSEE
INGHAM
LIVINGSTON
WASHTENAW
OTTAWA
ALLEGAN
BARRY
EATON
CALHOUN
JACKSON
10 0 10 20 30
SCALE OF MILES
LOCATION MAP

Figure VII-1.
Gauging Stations

statistics only are available for some periods. The records are considered good by the U.S.G.S., although they report occasional regulation at low flow by a mill sixteen miles upstream from the station. The average discharge at this station is 204 cfs. The maximum discharge for the period of records, 5,940 cfs, occurred on April 20, 1975. The minimum discharge of 3 cfs occurred on July 31, 1931.[6]

U.S.G.S. gauging station number 04111000, Grand River near Eaton Rapids, is located two miles northeast of Eaton Rapids. It is 400 feet upstream from the bridge on Petrieville Highway. This is 25 miles upstream from the confluence of the Red Cedar and Grand Rivers, and at mile 178 from the mouth of the Grand River. This station is approximately sixteen miles upstream of the boundary of the Lansing urban area. The period of record is from October, 1950 to the present year. The records are considered good by the U.S.G.S., except for the winter period which are considered fair. The average discharge at this station is 457 cfs. The maximum discharge for the period of record, 3,500 cfs, occurred on February 21, 1971. The minimum discharge of 14 cfs occurred on December 20, 1962.[6]

The flows of the Grand and Red Cedar Rivers are influenced by a series of dams and impoundments. Two affect the flow of the Red Cedar in East Lansing, one of these is the Williamston dam which is roughly sixteen miles upstream from the gauging station. This impoundment has a normal storage volume of 50 acre feet and a maximum storage volume of 60 acre feet. The second impoundment, the M.S.U. dam, has a normal storage of less than 50 acre feet. There are two dams on the Grand River in the City of Lansing, the Moores Park Dam and the North Lansing Dam. The Moores Park Dam is upstream of the confluence of the Grand and Red Cedar, it has a normal storage volume of 1,250 acre feet and a maximum storage of 1,550 acre feet. The North Lansing Dam is located downstream of the confluence of the Grand and Red Cedar Rivers and has a normal storage of 360 acre feet, with a maximum storage of 410 acre feet. Impoundments on the Grand River upstream from Lansing include the Smithville, Eaton Rapids, and Dimondale Dams. The farthest upstream is the Smithville Dam, which has a normal storage volume of 180 acre feet and a maximum of 210 acre feet. The Eaton Rapids Dam has a normal storage of 70 acre feet, with a 90 acre feet maximum. The Dimondale Dam has a normal storage of 50 acre feet and a maximum of 70 acre feet.

There is no flow data available for the Sycamore Creek in Lansing. Sycamore Creek is a tributary of the Red Cedar River, its flow is added downstream of the East Lansing gauging station. It drains areas consisting predominantly of agricultural, pasture, and forested areas. Based on five years of data collected near Holt, the flow at the mouth of the Sycamore Creek is estimated to be roughly 20-30% of the flow of the Red Cedar.[1]

The annual discharge statistics for the three stations are presented in Tables VII-1 through VII-3. Monthly discharge statistics for these stations are shown in Tables VII-4 through VII-6. Recurrence intervals and probability

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TABLE VII-1

Grand River at Eaton Rapids
Annual Discharge Statistics
For the Period of Record*

Mean Flow (cfs)	454
Standard Deviation	157
Coefficient of Variation	0.35

Flow Exceeded 'x' Percent of Time

x	Flow (cfs)
95	89
90	110
75	160
70	180
50	300
25	600
10	1,000

* Data for the years 1951 through 1979 were used.

TABLE VII-2

Red Cedar River at East Lansing
Annual Discharge Statistics
For the Period of Record*

Mean Flow (cfs)	201
Standard Deviation	88.3
Coefficient of Variation	0.44

Flow Exceeded 'x' Percent of Time

x	Flow (cfs)
95	19
90	26
75	44
70	50
50	87
25	220
10	490

* Data for the years 1903 and 1931 through 1979 were used.

TABLE VII-3

Grand River at Lansing Annual Discharge Statistics For the Period of Record*

Mean Flow (cfs)	824
Standard Deviation	295
Coefficient of Variation	0.36

Flow Exceeded 'x' Percent of Time

x	Flow (cfs)
95	130
90	170
75	260
70	290
50	480
25	990
10	1,900

* Data for the years 1902 through 1906 and 1935 through 1979 were used.

TABLE VII-4

Grand River at Eaton Rapids
Monthly Discharge Statistics for
The Period of Record*

	Mean (cfs)	Standard Deviation	Coefficient of Variation
October	228	151	0.66
November	321	156	0.49
December	423	206	0.49
January	463	342	0.74
February	549	341	0.62
March	914	378	0.41
April	925	303	0.33
May	632	327	0.52
June	383	237	0.62
July	266	223	0.84
August	186	118	0.64
September	181	150	0.83

* Data for the years 1951 through 1980 were used.

TABLE VII-5

Red Cedar River at East Lansing
Monthly Discharge Statistics
For the Period of Record*

	Mean (cfs)	Standard Deviation	Coefficient of Variation
October	77.8	85.6	1.10
November	109	83.7	0.77
December	158	129	0.82
January	193	186	0.96
February	272	236	0.87
March	498	282	0.57
April	467	294	0.63
May	290	255	0.88
June	162	128	0.79
July	83.0	97.1	1.17
August	50.6	36.6	0.72
September	61.3	73.8	1.20

* Data for the years 1903, 1904 and 1931 through 1979 were used.

TABLE VII-6

Grand River at Lansing
Monthly Discharge Statistics for The Period of Record*

	Mean (cfs)	Standard Deviation	Coefficient of Variation
October	368	258	0.70
November	490	272	0.56
December	632	385	0.61
January	746	587	0.79
February	959	659	0.69
March	1,919	1,120	0.58
April	1,752	880	0.50
May	1,121	751	0.67
June	810	604	0.75
July	478	417	0.87
August	324	203	0.63
September	320	262	0.82

* Data for the years 1901 through 1906 and 1935 through 1980 were used.

of occurrence calculations for various durations of drought flow are shown for each station in Tables VII-7 through VII-9. The average daily discharges for the three stations, from October of 1979 to September of 1981, are presented in pages 1 through 6 of Appendix III. This data was obtained from the U.S.G.S. Lansing office, and from Ref. 6. The statistical hydrologic data from the U.S.G.S. is also presented in Appendix III in its unsummarized version.

The physical characteristics of the rivers at each of the gauging stations are shown in Tables VII-10 through VII-12. River width, cross-sectional area, and average velocity are shown for a range of flows. This data was taken from U.S.G.S. field measurements. Although each of the gauging stations is located at or near a bridge on the river, cross-sectioning and velocity measurements are taken at a sufficient distance to prevent influences by the structures.

The Michigan Department of Natural Resources has detailed physical data on the Grand River from a survey conducted in August of 1970. River cross-sections were taken at 500 foot intervals from Dimondale (approximately two miles upstream of the Lansing urban area) to Jones Road in Grand Ledge (approximately 13 miles downstream of the Lansing urban area). In addition to area, average stream depth, width, and the volume between stations were also reported. A copy of this data, in field-note form, is available on request from ENCOTEC.

TABLE VII-7

Grand River at Eaton Rapids
Probabilities of Droughts for 7, 14, 30, 60 & 90 Consecutive Days

Rank m	Recurrence Interval, T (Years) T=(n+1)/m	Probability, P (%) P=100/T	Lowest Mean Discharge, in Increasing Order for the Following Number of Consecutive Days (cfs) 7	14	30	60	90
1	29	3.4	52	55	38	63	67
2	14.5	6.9	59	65	73	83	84
3	9.7	10.3	63	69	74	86	97
4	7.3	13.7	68	77	80	88	99
5	5.8	17.2	70	77	82	91	100
6	4.8	20.8	73	77	85	94	105
7	4.1	24.4	77	79	87	105	109
8	3.6	27.8	79	81	95	105	113
9	3.2	31.3	83	89	96	107	119
10	2.9	34.5	83	90	99	109	119
11	2.6	38.5	87	94	102	111	122
12	2.4	41.7	89	97	104	112	124
13	2.2	45.5	91	98	104	121	137
14	2.1	47.6	91	101	109	128	138
15	1.9	52.6	96	104	111	130	146
16	1.8	55.6	97	105	116	131	150
17	1.7	58.8	103	106	118	135	150
18	1.6	62.5	105	108	121	138	158
19	1.5	66.7	106	111	128	143	166
20	1.45	69.0	113	127	134	144	167
21	1.38	72.5	119	127	135	144	169
22	1.32	75.8	122	128	142	165	189
23	1.26	79.4	124	138	142	183	199
24	1.21	82.6	124	141	157	185	248
25	1.16	86.2	139	150	159	185	253
26	1.12	89.3	144	152	162	203	290
27	1.07	93.5	157	167	192	248	302
28	1.04	96.2	234	246	256	286	303

NOTE: Data for the years 1952 through 1979 were used.

TABLE VII-8

Red Cedar River at East Lansing
Probabilities of Droughts for 7, 14, 30. 60 & 90 Consecutive Days

Rank m	Recurrence Interval, T (Years) T=(n+1)/m	Probability, P (%) P=100/T	Lowest Mean Discharge, in Increasing Order for the Following Number of Consectutive Days (cfs) 7	14	30	60	90
1	49	2.0	3.9	4.0	5.6	7.5	9.4
2	24.5	4.1	6.7	7.9	9.2	12	15
3	16.3	6.1	7.5	9.6	13	14	15
4	12.3	8.1	7.7	11	13	17	18
5	9.8	10.2	9.8	12	14	17	18
6	8.2	12.2	10	13	14	17	18
7	7.0	14.3	11	13	15	17	19
8	6.1	16.4	11	14	16	19	21
9	5.4	18.5	11	14	17	19	22
10	4.9	20.4	12	14	17	20	22
11	4.5	22.2	12	17	17	22	25
12	4.1	24.4	13	17	20	23	26
13	3.8	26.3	14	17	21	24	27
14	3.5	28.6	14	18	23	25	28
15	3.3	30.3	15	19	23	26	28
16	3.1	32.3	16	20	23	28	29
17	2.9	34.5	17	21	24	28	31
18	2.7	37.0	17	22	24	29	33
19	2.6	38.5	18	22	24	29	34
20	2.45	40.8	20	22	25	30	35
21	2.33	42.9	20	24	26	31	35
22	2.23	44.8	20	24	26	31	37
23	2.13	46.9	21	24	27	32	37
24	2.04	49.0	22	24	28	33	38
25	1.96	51.0	22	24	30	34	40
26	1.88	53.2	22	24	30	35	40
27	1.81	55.2	23	26	31	37	41
28	1.75	57.1	25	26	31	37	43
29	1.69	59.2	25	27	31	38	43
30	1.63	61.3	26	28	33	38	44
31	1.58	63.3	26	28	33	39	44
32	1.53	65.4	26	29	33	39	46
33	1.48	67.6	27	29	34	39	46
34	1.44	69.4	27	30	34	40	46
35	1.40	71.4	27	30	34	42	52
36	1.36	73.5	29	32	35	42	57
37	1.32	75.8	29	32	36	43	58
38	1.29	77.5	30	32	38	48	58
39	1.26	79.4	31	33	38	51	60
40	1.23	81.3	34	35	39	52	61
41	1.20	83.3	38	39	42	52	67
42	1.17	85.5	38	40	42	53	70
43	1.14	87.7	38	41	45	53	75
44	1.11	90.1	39	42	48	68	81
45	1.09	91.7	39	43	51	75	83
46	1.07	93.5	42	44	55	76	93
47	1.04	96.2	48	49	61	78	109
48	1.02	98.0	61	64	69	86	130

NOTE: Data for the years 1932 through 1979 were used.

TABLE VII-9

Grand River at Lansing
Probabilities of Droughts for 7, 14, 30, 60 & 90 Consecutive Days

Rank m	Recurrence Interval, T (Years) T=(n+1)/m	Probability, P (%) P=100/T	Lowest Mean Discharge, in Increasing Order for the Following Number of Consecutive Days (cfs) 7	14	30	60	90
1	50	2.0	44	49	57	73	92
2	25	4.0	45	71	81	86	95
3	16.7	6.0	70	79	89	95	108
4	12.5	8.0	75	82	92	102	123
5	10	10.0	76	84	94	111	127
6	8.3	12.0	78	87	94	121	127
7	7.1	14.1	78	89	109	122	139
8	6.3	15.9	92	103	110	124	147
9	5.6	17.9	95	105	118	129	148
10	5.0	20.0	99	111	119	134	153
11	4.5	22.2	102	113	123	141	158
12	4.2	23.8	103	114	127	144	166
13	3.8	26.3	104	116	131	153	168
14	3.6	27.8	108	120	135	161	170
15	3.3	30.3	114	123	139	165	184
16	3.1	32.3	115	124	153	166	187
17	2.9	34.5	125	129	154	168	201
18	2.8	35.7	127	131	156	174	201
19	2.6	38.5	127	142	158	177	207
20	2.5	40.0	127	147	158	184	215
21	2.4	41.7	133	154	159	195	226
22	2.3	43.5	146	156	161	197	227
23	2.2	45.5	147	160	175	212	229
24	2.1	47.6	152	163	175	212	233
25	2.0	50.0	157	164	185	214	243
26	1.9	52.6	156	166	193	220	245
27	1.85	54.1	162	171	194	221	245
28	1.79	55.9	162	171	197	226	257
29	1.72	58.1	164	174	197	239	264
30	1.67	59.9	167	175	201	247	282
31	1.61	62.1	168	180	204	247	283
32	1.56	64.1	171	190	207	248	292
33	1.52	65.8	176	190	208	248	296
34	1.47	68.0	182	194	209	252	306
35	1.43	69.9	184	195	220	266	309
36	1.39	71.9	185	196	221	266	337
37	1.35	74.1	191	201	225	271	338
38	1.32	75.8	194	211	229	281	339
39	1.28	78.1	204	213	234	282	355
40	1.25	80.0	206	219	239	292	373
41	1.22	82.0	206	222	244	328	377
42	1.19	84.0	210	228	273	344	405
43	1.16	86.2	215	239	276	355	411
44	1.14	87.7	227	241	285	405	439
45	1.11	90.1	232	278	298	423	466
46	1.09	91.7	260	305	355	428	512
47	1.06	94.3	312	318	379	505	570
48	1.04	96.2	333	350	400	575	598
49	1.02	98.0	358	370	473	578	746

NOTE: Data for the years 1902 through 1906, and 1936 through 1979 were used.

The Michigan DNR has also conducted time-of-passage studies on the Grand and Red Cedar Rivers in Lansing. Data for six portions of the Red Cedar River from Harrison Road (4.5 miles upstream from the mouth) to the river mouth has been obtained from a 1960 cross-sectioning study, and 1970 and 1981 dye studies. The river was re-channelized in 1972 and the flow data for the 1981 study was preliminary, subject to revision by the U.S.G.S., so the time-of-passage curves generated by the DNR are not included in this report. They are available from ENCOTEC on request.

Capital Consultants, Inc. of Lansing have reported some time of passage data in their report on combined sewer overflows in the City of Lansing.[7] Time of passage, depth, area, and velocity are reported for nine segments of the Red Cedar from the river mouth to a distance of four miles upstream. This data, along with their description of the method of data collection, is presented in pages 7 through 9 of Appendix III.

Time of passage studies for reaches of the Grand River from a half-mile upstream of South Waverly Road (mile 158 from the Grand River mouth) to Bridge Street in Grand Ledge (roughly 6 miles downstream from the urban Lansing boundary) were conducted by the Michigan DNR. Time of passage curves were established based on dye studies conducted in 1970 and 1981. The curves are not presented in this report becaue the flow data was preliminary and subject to revision by the U.S.G.S. The data is available on request from ENCOTEC.

WASTE LOAD SOURCES

Waste load inputs to the Grand and Red Cedar Rivers consist of municipal wastewater treatment plant effluents, industrial water discharges, and combined sewer overflows in addition to urban stormwater runoff and other non-point source discharges. The sources whose discharge points are within the boundaries of the Lansing/East Lansing urban areas are summarized here. As previously stated, the upstream boundaries of the urban area on the Grand and Red Cedar Rivers are assumed to be Creyts (mile 162) and Okemos (mile 8.5) Roads, with Webster Road (mile 146) the downstream boundary on the Grand River.

Copies of the monthly discharge summaries for the Lansing, East Lansing, and Delta Township wastewater treatment plants were obtained from the Michigan Department of Natural Resources for the time period from October, 1979 to September of 1981. The data is generally reported to the DNR as a monthly average of daily values. An average of these monthly averages was calculated for each of the monitored parameters from each of the treatment plants. This data is presented in Table VII-13. The discharge of the Delta Township wastewater treatment plant is actually downstream from the defined urban boundary (approximately 0.25 miles), but because of its proximity is included in this table. In addition to these facilities, there are several municipal wastewater treatment plants or stabilization lagoons upstream of the defined urban area on each of the Grand and Red Cedar Rivers and Sycamore Creek.

TABLE VII-13

Average Discharge Characteristics of Lansing Area Wastewater Treatment Plants

Parameter	East Lansing[1]	Lansing[2]	Delta Township[3]
Discharge (MGD)	10.4	24.8	2.8
BOD_5-Max. (mg/l)	4.3	6.5	11.1
pH-Max	7.7	7.45	7.9
pH-Min.	7.1	6.6	7.15
D.O.-Min.	5.1	6.35	---
Suspended Solids (mg/l)	2.0	2.5	11.4
Phosphorus-Total (mg/l)	0.90	0.62	0.66
Ammonia Nitrogen (mg/l)	0.26	1.2	---
Fecal Coliforms (per 100 ml)	25	5	41

1. Discharges at mile 4 from Red Cedar mouth.
2. Discharges at mile 150 from Grand River mouth.
3. Discharges at mile 145.7 from Grand River mouth.

314
~~316~~

The industrial inputs in the study area are listed in Table VII-14 along with the type of water discharged. The data was taken from Tri-County Region 208 Water Quality Management Plan - Point Source Discharges. This publication contains data on the quantity and quality of the discharge streams, along with projections of future characteristics of the stream. All of the inputs in Table VII-14 are discharged to the Grand River, except for the Lansing Supply Co. which discharges to the Red Cedar.[8]

The City of Lansing's sewer system dates back to the mid-1800's and is served to a considerable extent by combined sewers. The total area served by the Lansing wastewater collection system is about 22,000 acres. About 10,800 acres are served by combined sewer systems. Much of the area served by combined sewers is in the older, central part of the city.

The distinction between separate and combined sewers is not always well defined. There are areas in Lansing where storm runoff is picked up in separate storm sewers, but is discharged into a combined sewer before overflowing into an open water course. Similarly, there are cases where sanitary flows from separated areas discharge into combined sewers and, therefore, overflow into a stream through regulators during storms. The City has adopted, for several years, a policy of installing completely separated sewer systems in unsewered areas and separating combined sewer areas where possible. It is assumed that the City will continue to follow this policy.

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TABLE VII-14

Industrial Wastewater Inputs in the Study Area

Name	Activity	Effluent
Lansing Supply Co.	Plumbing and Heating Supply Outlet	Cooling Water
Oldsmobile Plants 2 and 3	Automotive Component Machining. Bumper Plating	Cooling Water Cooling Tower Blowdown Air-Scrubber Water
Board of Water and Light Ottawa St. Station	Steam Generation Electricity Generation	Boiler Blowdown Condenser Cooling Water Floor Drains
Board of Water and Light Eckert Station	Electricity Generation Steam Generation	Condenser Cooling Water Treated Boiler Blowdown and Process Wastewater Discharge from Fly Ash Eductors
Lindell Drop Forge	Metal Forging	Contact Cooling Water
GMC Parts Division	Warehouse, Distribution Center	Non-contact Cooling Water
Board of Water and Light Erickson Station	Electricity Generation	Cooling System Blowdown Cooling System Overflow

In 1978 there were 51 active combined sewer overflow locations in the City of Lansing. A list of these locations is given in Appendix III.[8] The existing data on C50's in the study area consists of the reports to the City of Lansing by Capitol Consultants Inc., and McNamee, Porter and Seeley.[7,9] No corresponding data exists for East Lansing, Lansing Township, or Delta Township. Thus a detailed description of the CSO locations on the Grand and Red Cedar Rivers in the study area cannot be compiled.

WATER QUALITY DATA

Water quality data for the portions of the Grand and Red Cedar Rivers in the study area are reported in this section. The available data comes from ENCOTEC sampling as part of the NURP project, historical records from STORET, and river surveys conducted by the Michigan Department of Natural Resources.

Data was retrieved through STORET for forty stations on the Grand and Red Cedar Rivers and Sycamore Creek. The data includes analyses of water, sediment, and fish flesh samples. The level of detail ranges from a few D.O. and temperature measurements at some stations to a broad data base of two hundred samples for extensive parameter lists at other stations.

Data for selected parameters at eight of the stations is graphically presented in Figures VII-2 through VII-13.

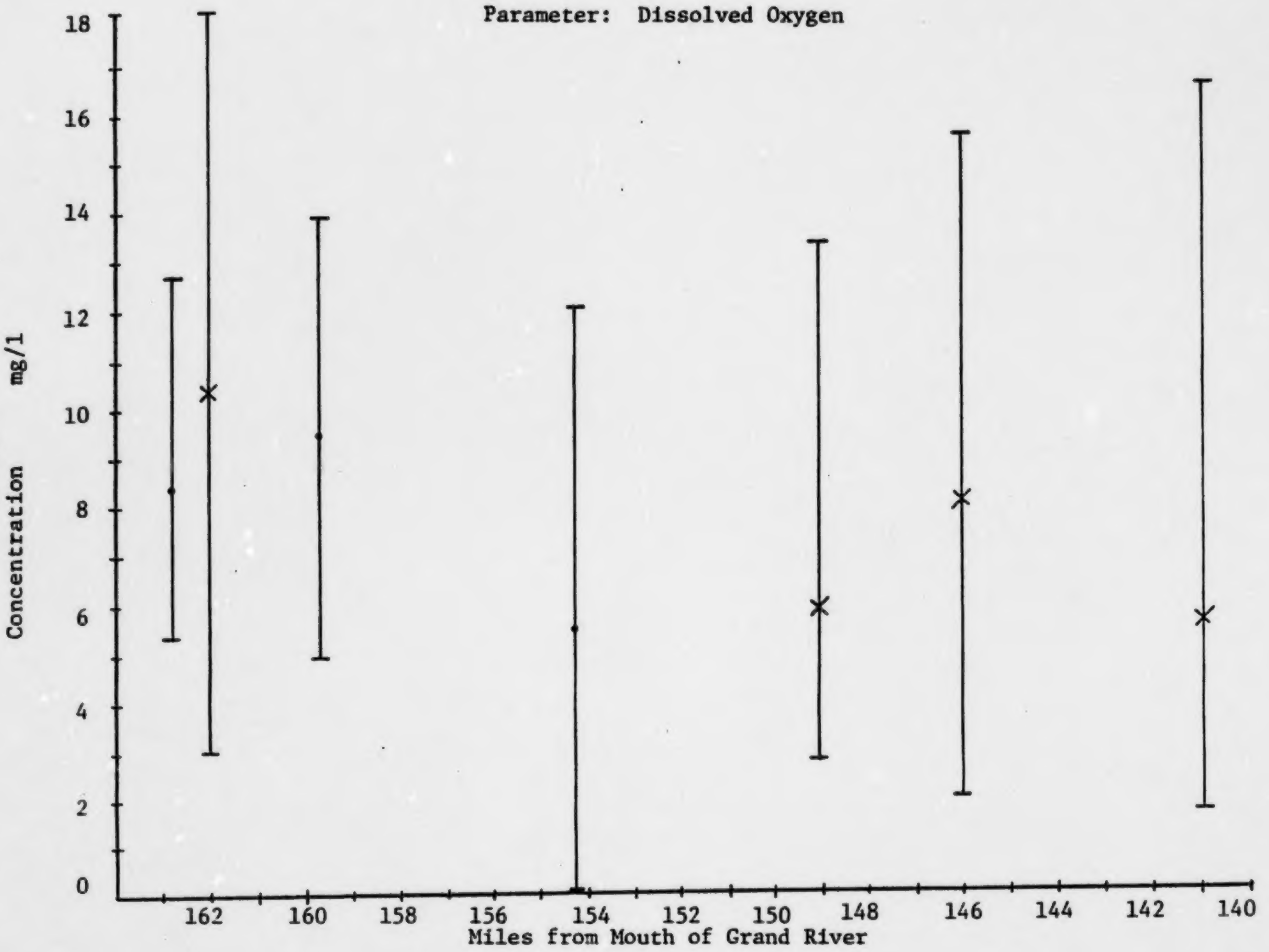


Figure VII-2. STORET Summary Data

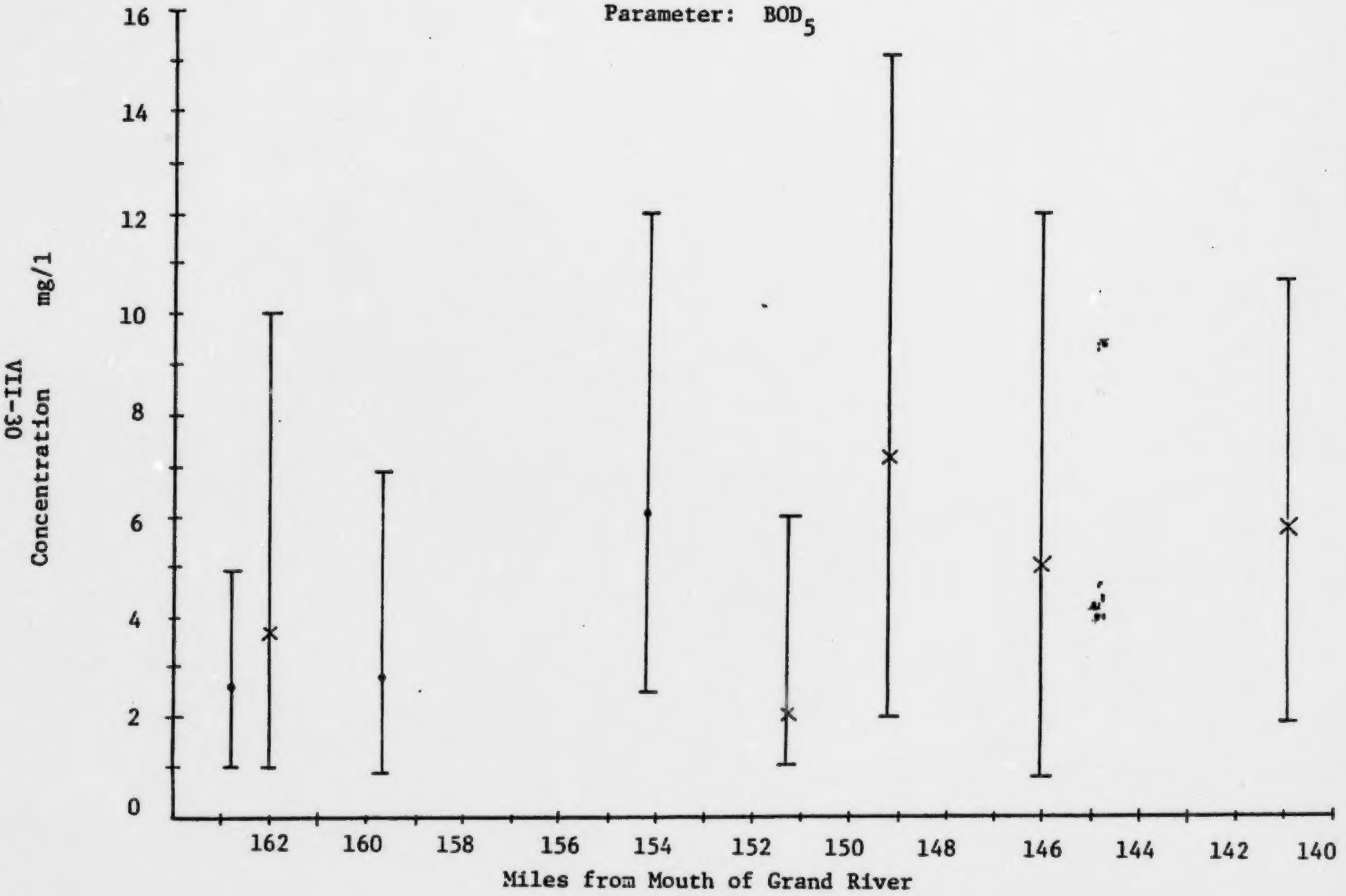


Figure VII-3. STORET Summary Data

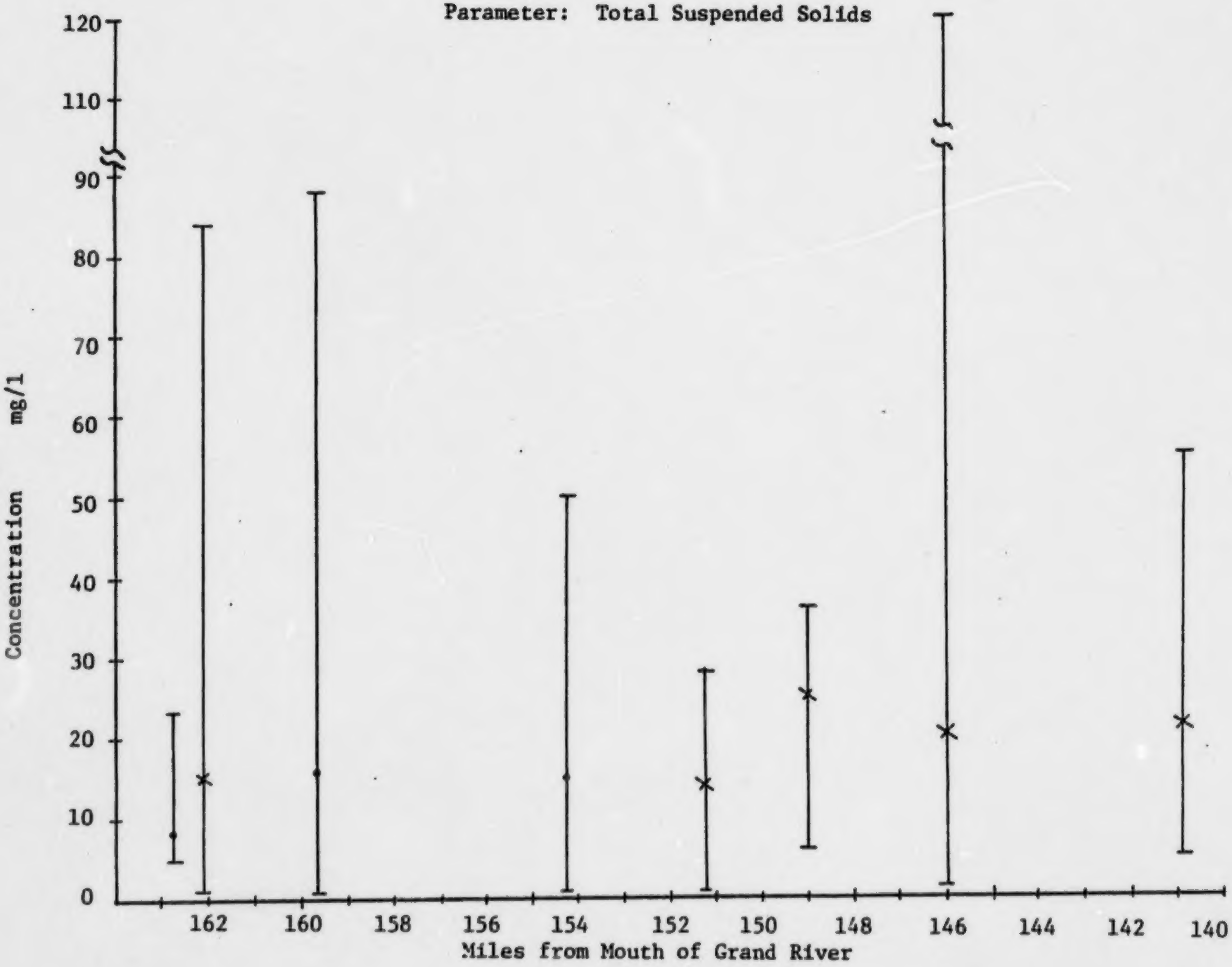


Figure VII-4. STORET Summary Data

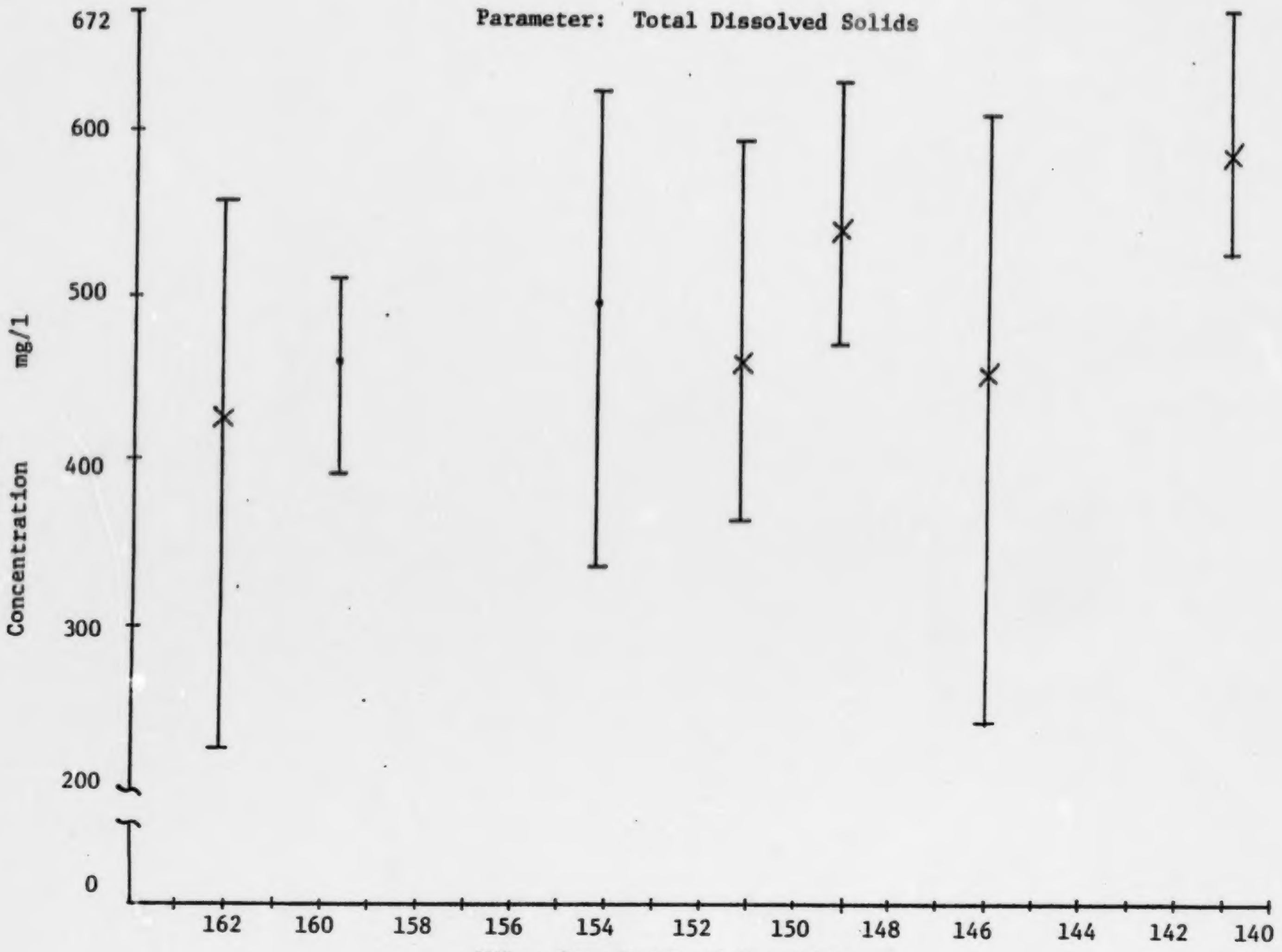


Figure VII-5. STORET Summary Data

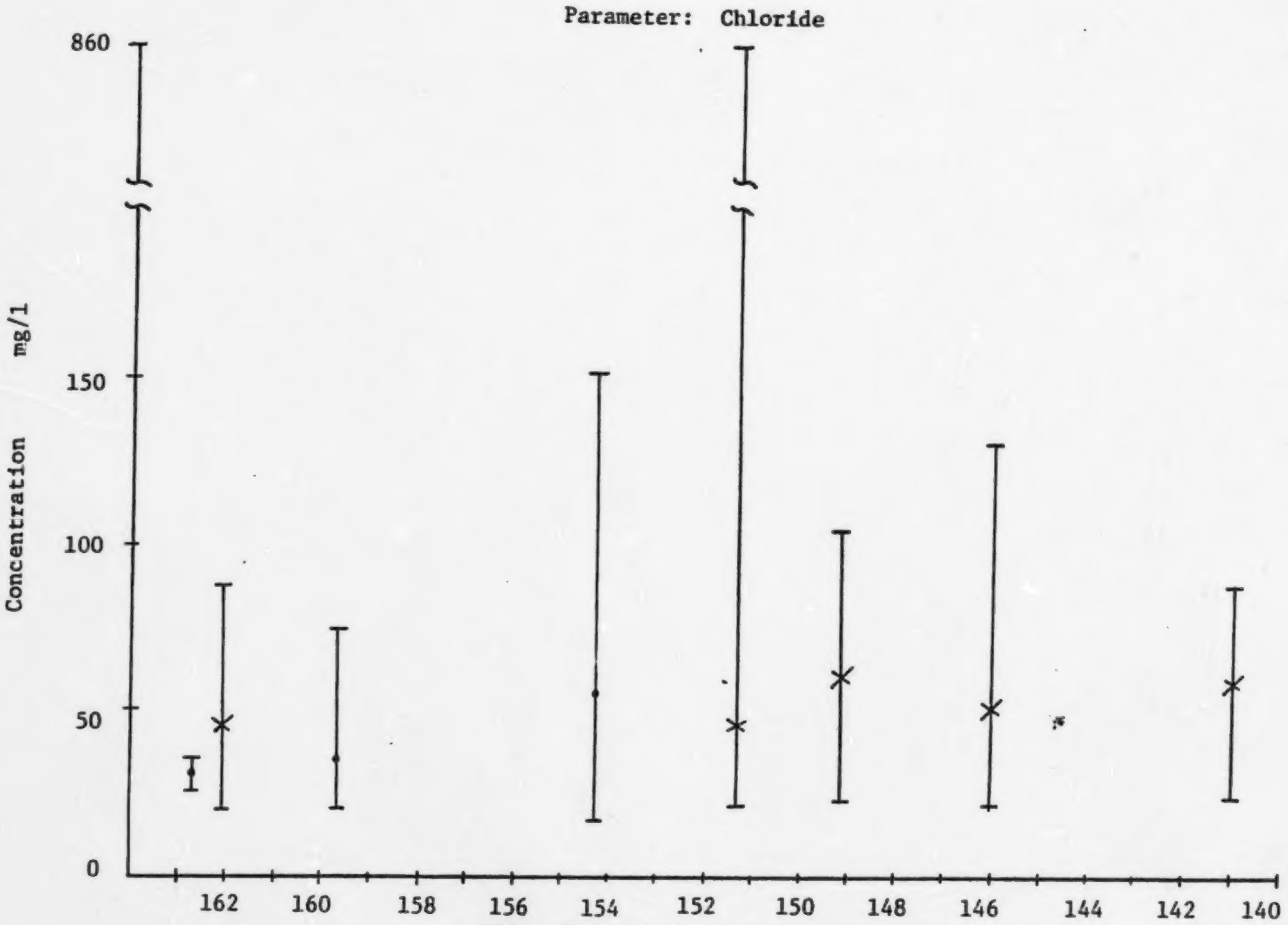


Figure VII-6. STORET Summary Data

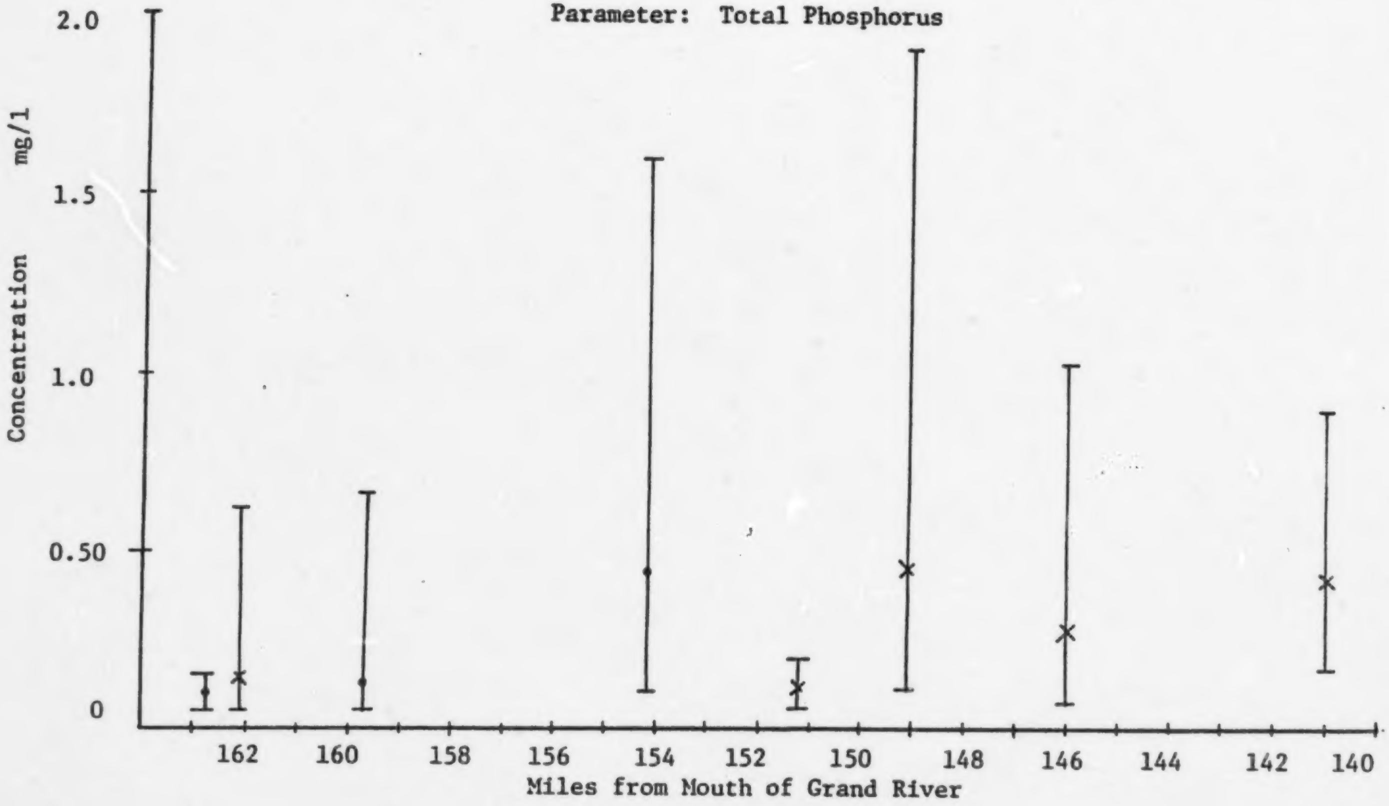

Parameter: Total Phosphorus

Figure VII-7. STORET Summary Data

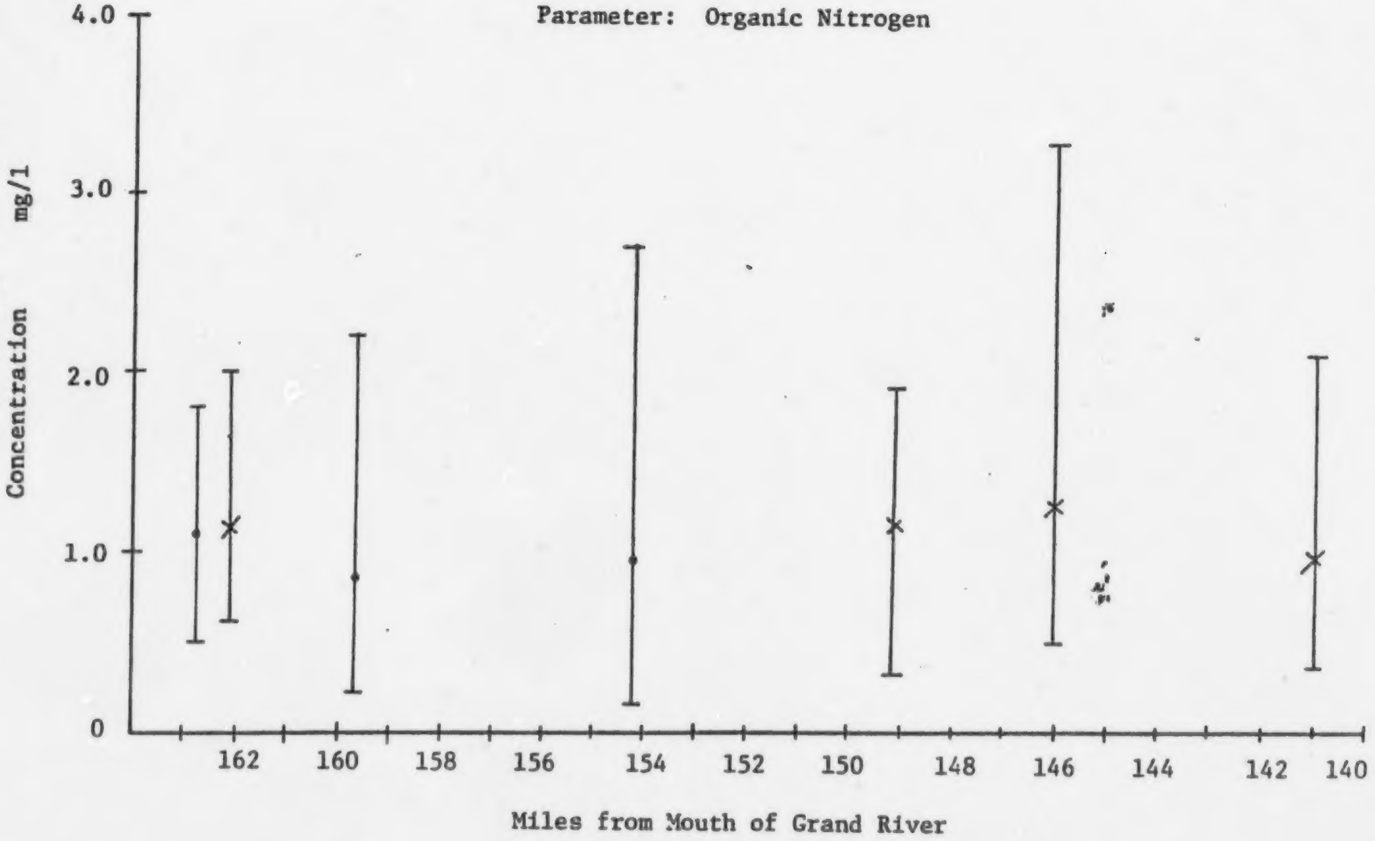
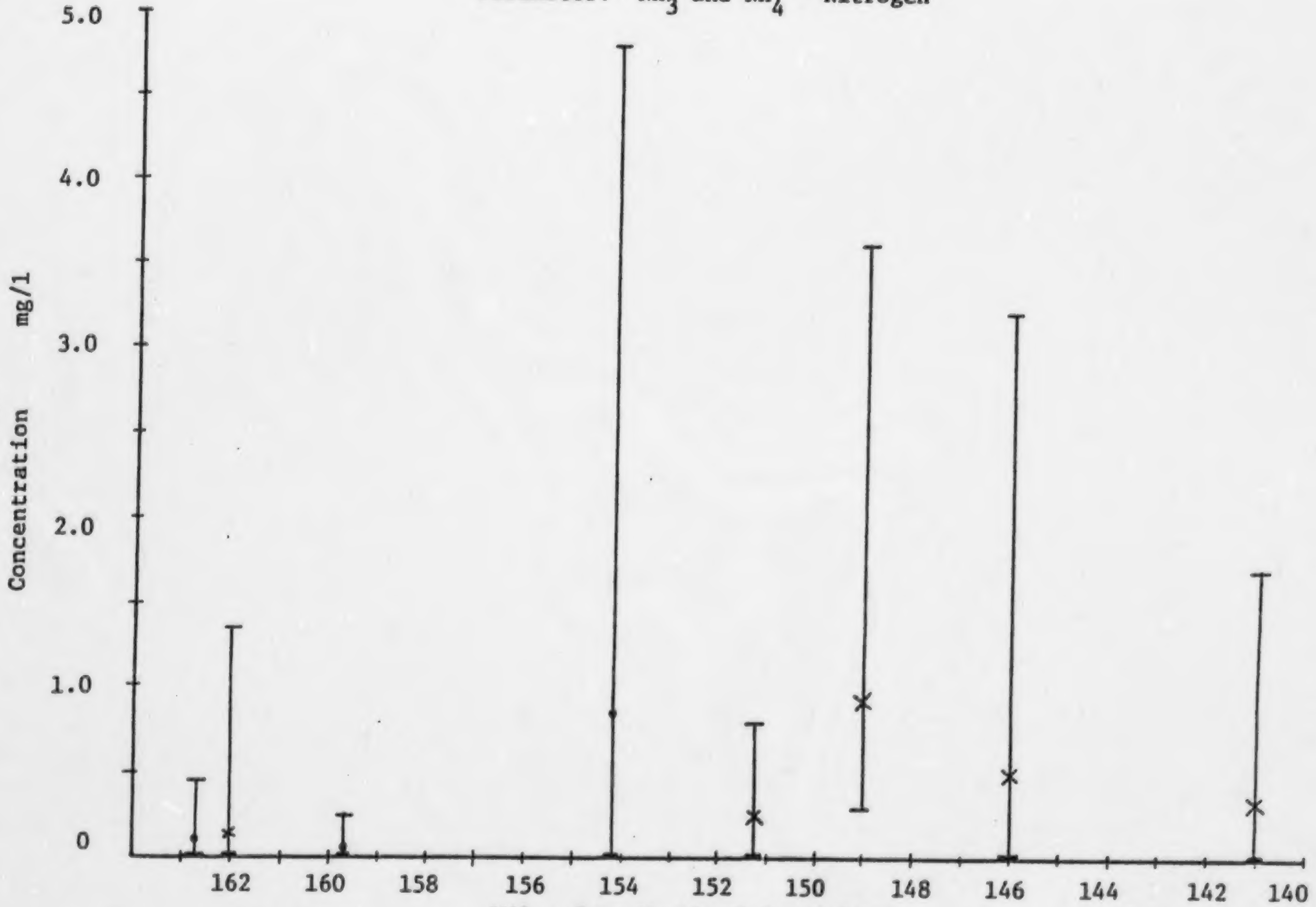


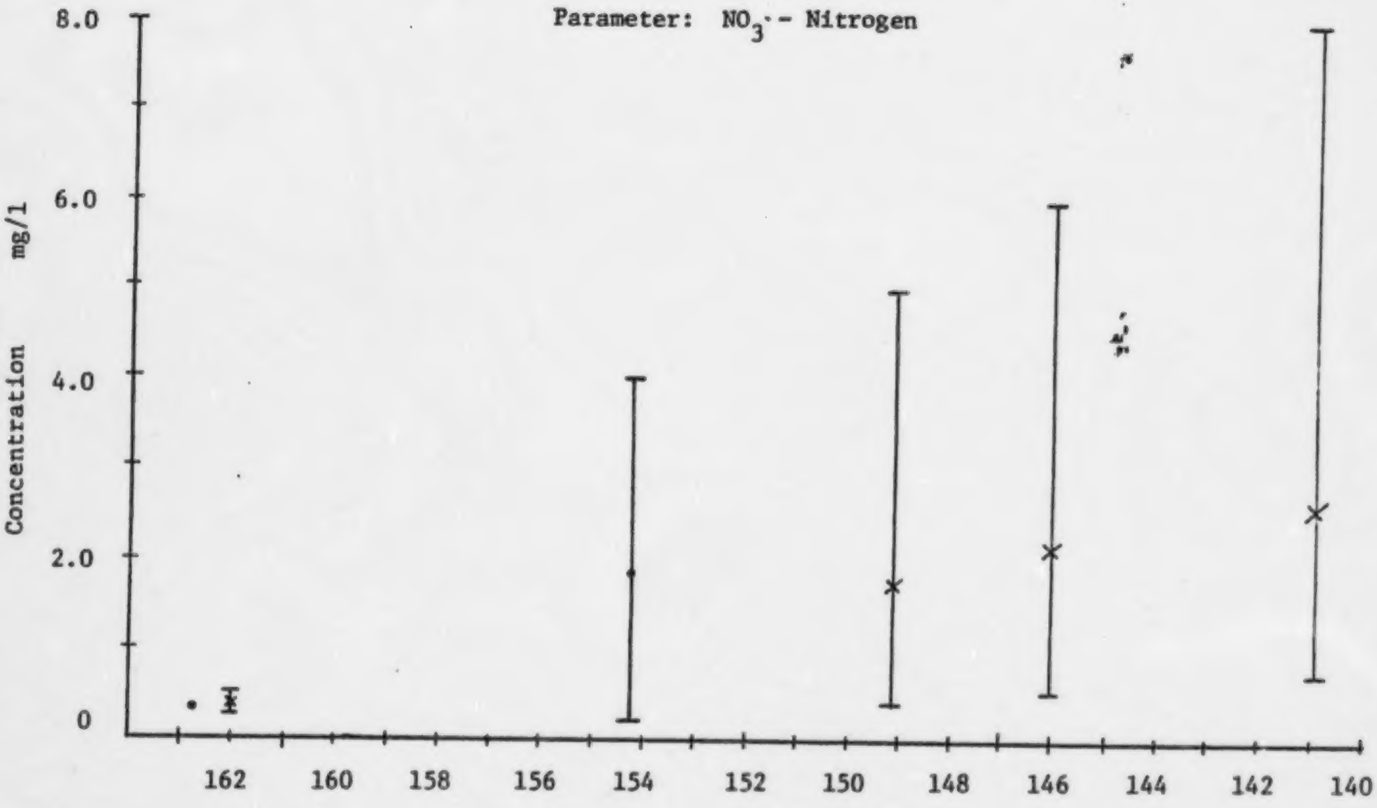
Figure VII-8. STORET Summary Data

Parameter: NH_3 and NH_4 - Nitrogen



Miles from Mouth of Grand River

Figure VII-9. STORET Summary Data



Miles from Mouth of Grand River

Figure VII-10. STORET Summary Data

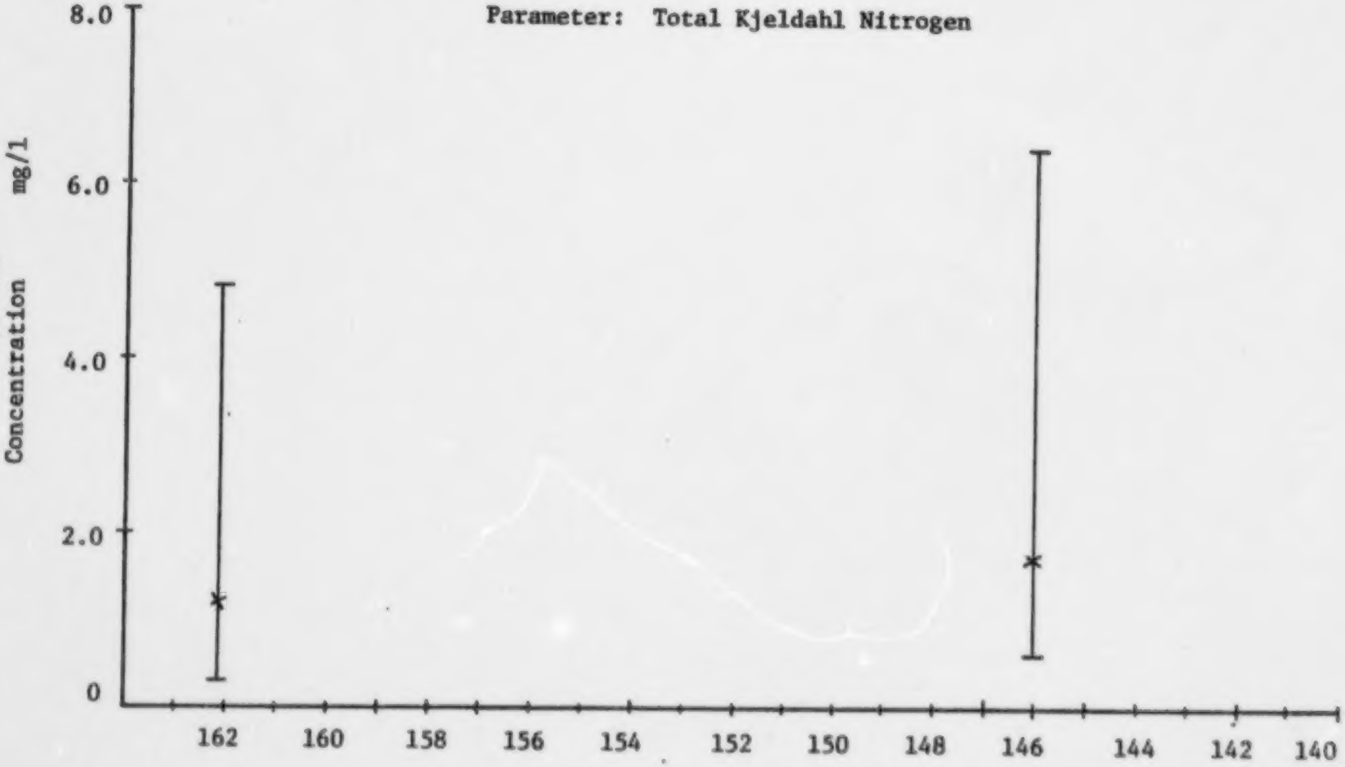


Figure VII-11. STORET Summary Data

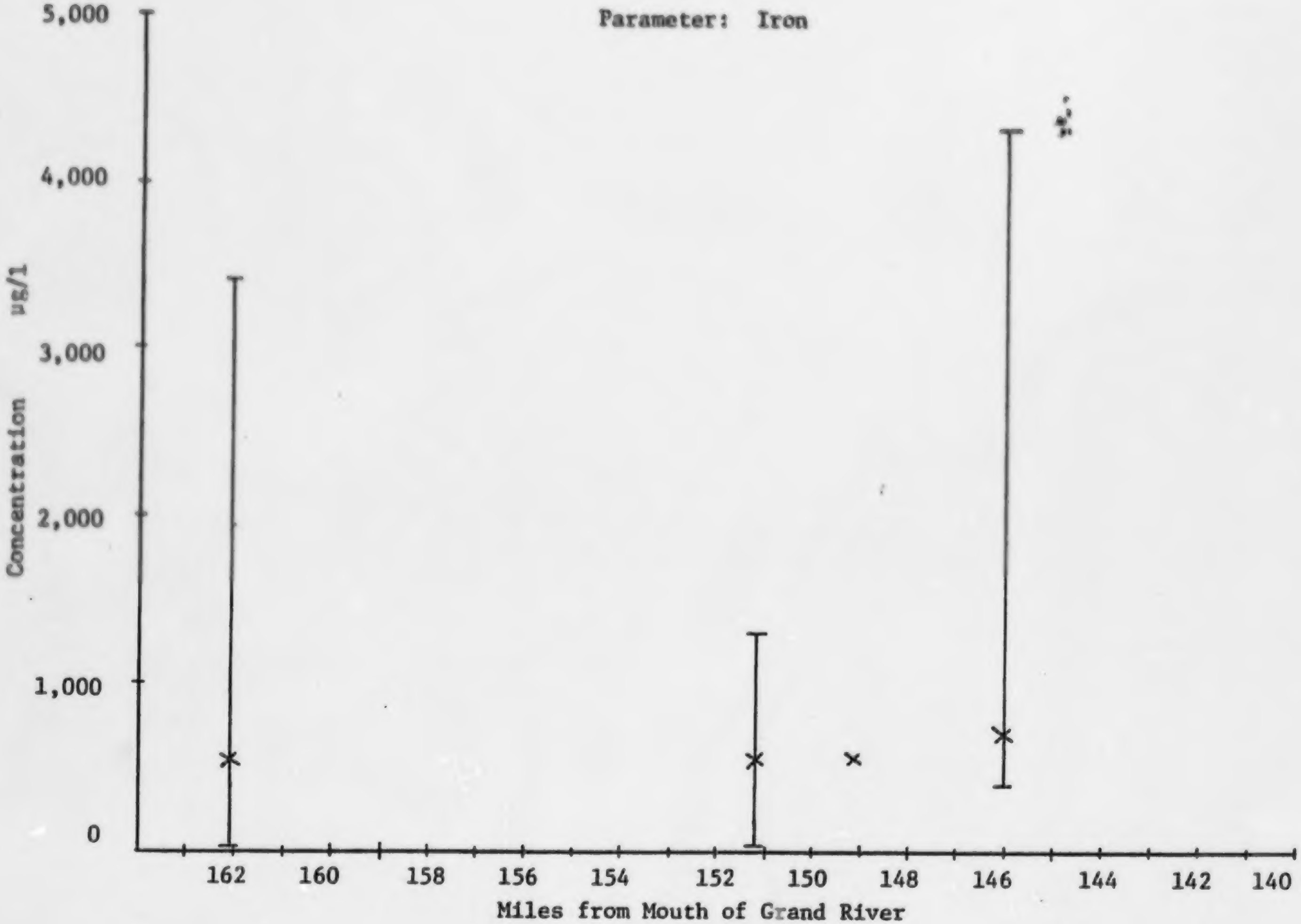


Figure VII-12. STORET Summary Data

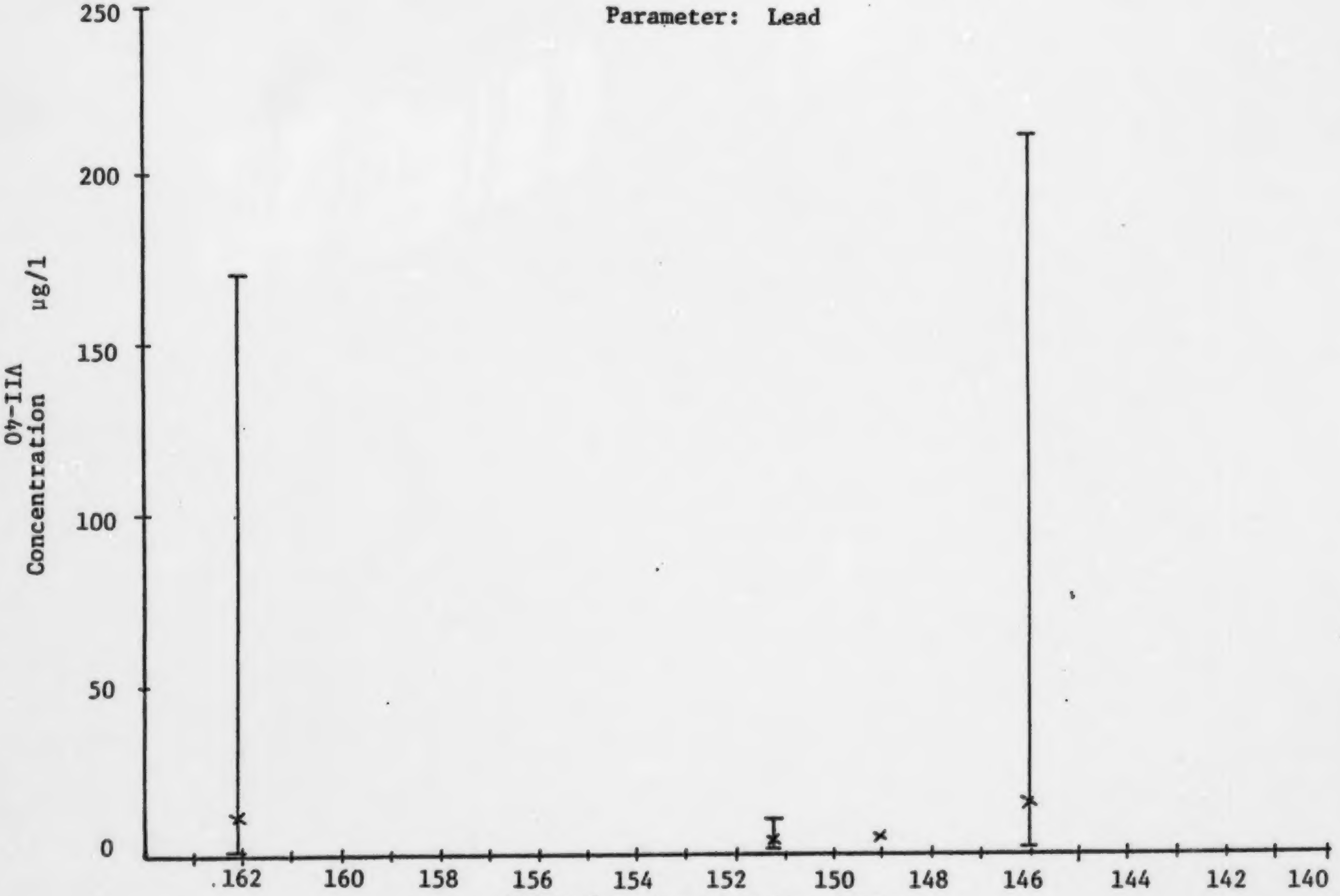


Figure VII-13. STORET Summary Data

The stations selected generally contained the greatest number of observations and the most detailed information. The time span over which the data was collected was roughly seven to thirteen years, except for the Logan Street and MSU stations where the data was collected over a two year period. The greatest quantity of data was collected at the Creyts and Webster Road stations, which the DNR samples monthly to evaluate the effects of the urban area on water quality. Aproximatley 100 samples were collected for nutrient analysis from the Red Cedar at MSU location over a two year period. The number of samples taken at the other stations range from about twenty to fifty. The names of the stations used for these figures, and the STORET number and river mile, are given on page 10 of Appendix III. The complete STORET output is contained in the Data Report.

In Figures VII-2 through VII-13, the maximum, minimum, and average values of the various parameters are presented. Stations on the Red Cedar are shown by bars showing the range of measured values with a dot for the average concentration. Grand River stations have the bar showing the range of values with a cross denoting the average concentration. Mile-points for the stations on the Red Cedar are determined by adding the mile-point from the mouth of the Red Cedar to the distance from the mouth of the Grand River to the confluence of the Grand and Red Cedar Rivers (154 miles). Significant mile-points to remember when viewing the figures are:

162.7 - Upstream boundary of the urban area on the Red Cedar:

162.1 - Upstream boundary of the urban area on the Grand River

158 - Discharge of the East Lansing WWTP to the Red Cedar

154 - Confluence of the Grand and Red Cedar Rivers

150 - Discharge of the Lansing WWTP to the Grand River

146 - Downstream boundary of the urban area on the Grand River

145.7 - Discharge of the Delta Township WWTP to the Grand River

The degradation of water quality by the urban area can easily be seen in these figures, as the average concentration of almost every pollutant parameter (with the possible exception of the metals) exhibits a significant increase. Violations of the D.O. standard (minimum of 4 mg/l) are seen at some point in time at five of the seven stations reporting D.O. data. The data does not indicate that violations of the average daily concentration of 5 mg/l are occurring. The total suspended solids guidelines of 25 mg/l appears to be frequently exceeded, especially at the Grand River stations farthest downstream. Two stations on the Grand River appear to be violating the 500 mg/l monthly average TDS standard, along with the Red Cedar station closest to the mouth. Occasional phosphorus concentrations in excess of the 0.5 mg/l guideline are seen at most of the stations, along with frequent occurences of ammonia nitrogen and total organic nitrogen concentraitons above the recommended guidelines of 0.4 mg/l and 0.8 mg/l, respectively.

Examination of this data set does not tell if the general water quality trends over a period of years are toward better or worse conditions. It is also impossible to tell which seasons of the year are most likely to cause problems. ENCOTEC requested STORET retrievals of the historical data sorted by individual seasons, and for individual years at the beginning and end of the period of record. DNR personnel replied that these types of retrievals are possible but because of time and cost constraints are generally not performed for anyone but government agencies. Thus, the data which was unavailable to ENCOTEC might be obtained by NURP project personnel if it was requested.

ENCOTEC collected samples from three locations on the Grand River once each month from November, 1979 to November of 1981. The samples were collected at the Creyts, Logan and Webster Road bridges. These locations were picked for determination of water quality upstream of, within, and at the downstream boundary of the urban area. The samples were collected on the same day that the MDNR collected their samples at the Creyts and Webster Road locations.

Initially, ENCOTEC analyzed the samples for all of the desired parameters, even though this duplicated the work of the MDNR. A comparison of six months of ENCOTEC and MDNR data showed good agreement of the reported values, so the parameter lists were adjusted to reduce the repitition of analyses. The water quality data for the Grand River station is shown in Table VII-15. No distinction is made between ENCOTEC and MDNR data in the later months of the program. The comparison of ENCOTEC and MDNR data is shown in Table VII-16.

Occasional violations of the suspended solids (25 mg/l) and ammonia nitrogen (0.4 mg/l) guidelines are seen in this data. Very low levels of the metals are observed. The highest lead concentration, 8 ug/l, is well below the 50 ug/l proposed guidelines for drinking water.

The Michigan Department of Natural Resources conducted several water quality surveys on the Grand River in the summer of 1981. The surveys conducted consisted of a 48-hour dry weather survey conducted from August 24 through 26, a 24-hour dry weather survey conducted on August 18 and 19, and a 24-hour wet weather survey conducted on July 7 and 8.

Only the data for the 48-hour dry weather survey had been entered into STORET at the time this report was written. Samples were collected at twenty-four stations from Creyts Road upstream of Lansing to M-100 in Grand Ledge. The survey data for selected parameters are summarized in Figures VII-14 through VII-20. The station names, STORET number, and river mile locations are listed on page 11 of Appendix III.

TABLE VII-15

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	November, 1979			December, 1979			January, 1980		
			C	L	W	C	L	W	C	L	W
I	Total Solids	mg/l	440	460	430	450	500	490	480	510	510
	Total Suspended Solids	mg/l	3	8	16	1	1	1	4	18	4
	pH	s.u.	8.03	8.06	7.93	8.01	8.07	8.05	7.75	7.98	7.88
	Total Alkalinity	mg/l	223	215	201	220	230	200	190	210	210
	Specific Conductance	umhos	745	---	790	660	680	710	670	680	670
	Chloride	mg/l	57	68	66	55	61	73	55	48	60
	Turbidity	NTU	2	4	6	5.4	2.8	3.2	4.2	3.6	3.2
III-S	Soluble Organic Carbon	mg/l	8	8	11	10	5	---	---	---	---
	Soluble Kjeldahl Nitrogen	mg/l	0.67	0.63	1.1	1.0	0.9	1.1	0.85	1.1	1.2
	Soluble Phosphorus	mg/l	0.06	0.06	0.14	0.06	0.06	0.17	0.05	0.06	0.12
III-T	TOC	mg/l	8	9	12	11	12	11	10	12	12
	Total Phosphorus	mg/l	0.11	0.13	0.22	0.07	0.09	0.19	0.11	0.10	0.16
	Ammonia Nitrogen	mg/l	0.64	0.79	0.74	0.40	0.35	0.23	0.27	0.30	0.28
	Total Kjeldahl Nitrogen	mg/l	0.83	0.79	1.3	1.0	0.9	1.2	1.5	2.2	1.6
	NO_3+NO_2-N	mg/l	1.1	1.4	2.3	0.92	1.7	2.6	1.4	1.7	2.5
	Grease and Oil	mg/l	8	5	18	1	1	2	2	1	2

C = Creyts
L = Logan
W = Webster

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TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	November, 1979 C	November, 1979 L	November, 1979 W	December, 1979 C	December, 1979 L	December, 1979 W	January, 1980 C	January, 1980 L	January, 1980 W
IV	BOD_5	mg/l	<1	2	4	1	1	1	1	1	<1
V	COD	mg/l	20	23	12	25	24	20	20	23	27
VI	Lead	mg/l	<0.001	0.002	0.002	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
	Iron	mg/l	0.18	0.20	0.20	0.16	0.17	0.14	0.20	0.17	0.15
	Zinc	mg/l	0.027	0.028	0.037	0.043	0.037	0.048	0.063	0.038	0.043
	Chromium	mg/l	0.009	0.008	0.013	0.017	0.031	0.026	0.017	0.031	0.015
	Copper	mg/l	<0.001	0.004	0.007	<0.001	0.005	0.009	<0.001	0.003	0.005
	Nickel	mg/l	0.040	0.025	0.035	0.030	0.017	0.034	0.017	0.014	0.024
	Cadmium	mg/l	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	0.0001	0.0001	0.0004	<0.0001
	Mercury	mg/l	<0.0002	<0.0002	0.0002	<0.0002	0.0025	<0.0002	<0.0002	<0.0002	<0.0002
	Arsenic	mg/l	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
VIII	Total Flouride	mg/l	0.24	0.27	0.32	0.20	0.20	0.27	0.17	0.20	0.25
IX	Orthophosphate	mg/l	0.08	0.10	0.18	0.06	0.06	0.15	0.04	0.05	0.11
X	Phenolics	mg/l	0.006	0.007	0.005	0.004	0.004	0.008	0.004	0.004	0.005
XI	Total Sulfide	mg/l	<0.01	0.01	<0.01	---	---	---	---	---	---
	Sulfate	mg/l	76	79	79	78	93	100	96	110	110
	Flow Rate	cfs	223**	---	273**	230**	---	312**	437**	---	674**

C = Creyts
L = Logan
W = Webster

** Data from U.S. Geological Survey.

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	February, 1980 C	February, 1980 L	February, 1980 W	March, 1980 C	March, 1980 L	March, 1980 W	April, 1980 C	April, 1980 L	April, 1980 W
I	Total Solids	mg/l	380	390	410	480	600	620	530	550	550
	Total Suspended Solids	mg/l	11	15	15	6	8	6	14	16	22
	pH	s.u.	7.84	7.79	7.80	7.93	8.03	8.04	8.17	8.22	8.24
	Total Alkalinity	mg/l	160	160	160	220	230	210	180	190	190
	Specific Conductance	umhos	460	490	520	660	630	660	510	520	550
	Chloride	mg/l	48	47	53	71	120	130	37	36	38
	Turbidity	NTU	7	12	12	6	8	5	5	6	5
III-S	Soluble Organic Carbon	mg/l	9	10	5	6	6	8	13	---	13
	Soluble Kjeldahl Nitrogen	mg/l	1.6	1.7	1.8	1.2	1.4	1.4	0.83	---	1.0
	Soluble Phosphorus	mg/l	0.07	0.02	0.05	0.07	0.09	0.16	0.03	0.12	0.05
III-T	TOC	mg/l	9	13	9	6	10	8	13	---	17
	Total Phosphorus	mg/l	0.13	0.19	0.24	0.11	0.14	0.22	0.11	0.13	0.16
	Ammonia Nitrogen	mg/l	0.69	0.80	0.83	0.68	0.69	0.68	0.03	---	0.21
	Total Kjeldahl Nitrogen	mg/l	1.6	1.7	1.8	1.6	1.6	1.7	1.0	1.2	1.4
	NO_3+NO_2-N	mg/l	1.4	2.4	2.5	1.5	1.7	3.2	1.8	---	1.8
	Grease and Oil	mg/l	2	2	2	1	2	2	1	---	2

C = Creyts
L = Logan
W = Webster

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	----February, 1980----- C	L	W	------March, 1980------ C	L	W	------April, 1980------ C	L	W
IV	BOD_5	mg/l	4	3	3	2	2	2	2.7	---	4.5
V	COD	mg/l	32	34	31	17	22	23	46	40	49
VI	Lead	mg/l	0.001	0.001	0.001	0.002	0.007	0.004	0.001	0.002	0.002
	Iron	mg/l	0.43	0.63	0.52	0.35	0.40	0.31	0.37	0.43	0.54
	Zinc	mg/l	0.05	0.04	0.04	0.048	0.057	0.055	0.033	0.029	0.035
	Chromium	mg/l	0.023	0.022	0.022	0.033	0.014	0.024	0.016	0.012	0.013
	Copper	mg/l	0.004	0.005	0.006	0.002	0.010	0.013	0.001	0.005	0.007
	Nickel	mg/l	0.022	0.016	0.030	0.025	0.018	0.033	0.016	0.012	0.018
	Cadmium	mg/l	0.0002	0.0001	<0.0001	<0.0001	0.0001	<0.0001	0.0002	0.0002	0.0011
	Mercury	mg/l	<0.0002	<0.0002	<0.0002	<0.0002	<0.0002	0.0002	0.0013	0.0008	0.0006
	Arsenic	mg/l	0.003	0.003	0.003	<0.001	0.001	0.001	0.001	0.001	0.001
VIII	Total Flouride	mg/l	---	---	---	---	---	---	---	---	---
IX	Orthophosphate	mg/l	0.069	---	0.142	0.138	---	0.173	0.008	---	0.041
X	Phenolics	mg/l	0.011	---	0.019	0.019	---	0.015	0.004	---	0.004
XI	Total Sulfide	mg/l	---	---	---	---	---	---	---	---	---
	Flow Rate	cfs	546**	---	893**	294**	---	468**	900**	---	1,360**

C = Creyts
L = Logan
W = Webster

** Data from U.S. Geological Survey.

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	May, 1980 C	May, 1980 L	May, 1980 W	June, 1980 C	June, 1980 L	June, 1980 W	July, 1980 C	July, 1980 L	July, 1980 W
I	Total Solids	mg/l	420	460	470	420	460	450	440	380	450
	Total Suspended Solids	mg/l	6	13	22	14	28	26	13	18	17
	pH	s.u.	8.19	8.02	8.15	8.14	8.17	8.10	8.31	8.31	8.33
	Total Alkalinity	mg/l	220	220	210	230	220	210	210	210	200
	Specific Conductance	umhos	630	670	680	580	550	590	250	610	660
	Chloride	mg/l	38	46	52	30	32	36	49	55	72
	Turbidity	NTU	5	5	6	6	16	16	5	9	7
III-S	Soluble Organic Carbon	mg/l	---	---	---	12	8	10	10	8	8
	Soluble Kjeldahl Nitrogen	mg/l	---	---	---	1.0	1.5	1.2	0.52	0.64	0.81
	Soluble Phosphorus	mg/l	---	---	---	0.08	0.07	0.08	0.03	0.06	0.07
III-T	TOC	mg/l	12	13	15	12	12	10	13	13	11
	Total Phosphorus	mg/l	0.08	0.10	0.17	0.13	0.16	0.16	0.07	0.09	0.12
	Ammonia Nitrogen	mg/l	0.08	0.16	0.13	0.11	0.13	0.11	0.07	0.14	0.20
	Total Kjeldahl Nitrogen	mg/l	0.99	1.3	1.6	1.2	1.6	1.5	0.90	1.1	2.3
	NO_3+NO_2-N	mg/l	1.2	0.76	1.4	0.97	3.2	3.6	0.78	2.1	1.6
	Grease and Oil	mg/l	1	1	2	1	1	1	2	2	4

C = Creyts
L = Logan
W = Webster

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	--------May, 1980------- C	L	W	------June, 1980------- C	L	W	------July. 1980------- C	L	W
IV	BOD	mg/1	2	3	5	10	6	2	4	4	3
V	COD	mg/1	35	40	44	44	47	47	30	31	33
VI	Lead	mg/1	<0.001	0.001	0.004	<0.001	0.001	0.002	0.001	0.002	0.003
	Iron	mg/1	0.45	0.52	0.68	0.71	0.95	1.0	0.30	0.39	0.36
	Zinc	mg/1	0.030	0.032	0.050	0.034	0.045	0.045	0.020	0.024	0.027
	Chromium	mg/1	0.031	0.052	0.015	0.014	0.009	0.010	0.009	0.007	0.005
	Copper	mg/1	0.001	0.004	0.008	0.002	0.004	0.007	0.002	0.005	0.007
	Nickel	mg/1	0.016	0.012	0.016	0.014	0.011	0.016	0.010	0.008	0.016
	Cadmium	mg/1	0.0001	0.0002	0.0002	<0.0001	<0.0001	<0.0001	0.0002	0.0002	0.0002
	Mercury	mg/1	<0.0002	<0.0002	<0.0002	<0.0002	<0.0002	<0.0002	---	---	---
	Arsenic	mg/1	0.001	0.002	0.002	0.003	0.003	0.003	---	---	---
VIII	Total Flouride	mg/1	---	---	---	0.13	0.12	0.27	0.25	0.30	0.33
IX	Orthophosphate	mg/1	0.033	---	0.048	0.02	0.03	0.04	0.04	0.08	0.08
X	Phenolics	mg/1	0.0016	---	0.0032	0.0032	---	0.0034	<0.001	<0.001	<0.001
XI	Total Sulfide	mg/1	---	---	---	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
	Flow Rate	cfs	510**	---	679**	658**	---	1,470**	163**	---	267**

C = Creyts
L = Logan
W = Webster

** Data from U.S. Geological Survey.

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	-----August, 1980------ C	L	W	----September, 1980---- C	L	W	-----October, 1980----- C	L	W
I	Total Solids	mg/l	460	480	470	483	470	484	510	490	555
	Total Suspended Solids	mg/l	14	24	20	15	18	16	9	9	15
	pH	s.u.	8.14	8.19	8.19	8.2	8.26	8.3	8.10	8.20	8.10
	Total Alkalinity	mg/l	210	220	200	215	220	199	250	480	245
	Specific Conductance	umhos	540	550	540	620	580	605	705	680	740
	Chloride	mg/l	55	53	53	37	41	37	40	51	44
	Turbidity	NTU	9	10	11	8	10	13	4	4	6
III-S	Soluble Organic Carbon	mg/l	11	15	16	14	11	15	20	18	18
	Soluble Kjeldahl Nitrogen	mg/l	0.81	0.87	0.83	0.98	0.87	1.0	0.75	0.87	1.1
	Soluble Phosphorus	mg/l	0.08	0.07	0.13	0.07	0.08	0.11	0.06	0.08	0.11
III-T	TOC	mg/l	11	17	18	15	16	17	20	24	22
	Total Phosphorus	mg/l	0.15	0.17	0.22	0.12	0.21	0.16	0.10	0.14	0.15
	Ammonia Nitrogen	mg/l	0.19	0.12	0.07	0.08	0.08	0.07	0.13	0.13	0.26
	Total Kjeldahl Nitrogen	mg/l	1.2	1.2	1.5	0.98	1.2	1.2	0.88	0.99	1.3
	NO_3+NO_2-N	mg/l	0.55	0.79	1.2	0.62	1.0	1.4	1.7	1.6	0.9
	Grease and Oil	mg/l	3	3	2	<1	1	<1	2	2	3

C = Creyts
L = Logan
W = Webster

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	-----August, 1980------ C	L	W	----September, 1980---- C	L	W	-----October, 1980----- C	L	W
IV	BOD	mg/l	4	5	4	3	2	3	<1	1	1
V	COD	mg/l	32	40	29	35	41	42	31	---	33
VI	Lead	mg/l	0.001	0.002	0.002	<0.005	0.010	<0.005	0.001	0.002	0.002
	Iron	mg/l	0.46	0.59	0.53	0.83	0.90	0.75	0.56	0.65	0.57
	Zinc	mg/l	0.029	0.033	0.036	0.03	0.06	0.02	0.06	0.06	0.06
	Chromium	mg/l	0.015	0.008	0.015	0.005	0.013	<0.005	0.006	0.008	0.005
	Copper	mg/l	0.002	0.006	0.008	<0.002	0.006	0.006	0.001	0.005	0.006
	Nickel	mg/l	0.014	0.010	0.020	0.017	0.012	0.017	0.006	0.011	0.012
	Cadmium	mg/l	<0.0002	<0.0002	0.0002	<0.002	<0.0002	<0.002	<0.0002	<0.0002	<0.0002
VII	Total Flouride	mg/l	0.18	0.20	0.08	0.18	0.18	0.20	0.61	0.23	0.16
IX	Orthophosphate	mg/l	0.05	0.05	0.10	0.060	0.04	0.079	0.056	0.05	0.103
X	Phenolics	mg/l	0.003	0.007	0.002	<0.0005	0.008	<0.0005	<0.0005	0.004	<0.C005
XI	Total Sulfide	mg/l	<0.01	<0.01	<0.01	---	---	---	<0.01	<0.01	<0.01
	Flow Rate	cfs	332	---	557	460	---	840	314	---	534

C = Creyts
L = Logan
W = Webster

** Data from U.S. Geological Survey.

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	November 1980 C	November 1980 L	November 1980 W	December, 1980 C	December, 1980 L	December, 1980 W	January, 1981 C	January, 1981 L	January, 1981 W
I	Total Solids	mg/l	473	---	501	404	---	471	544	480	526
	Total Suspended Solids	mg/l	5	---	9	8	---	3	4	1	6
	pH	s.u.	8.30	---	8.30	7.90	---	8.00	7.90	8.00	7.80
	Total Alkalinity	mg/l	260	---	265	240	---	245	265	260	255
	Specific Conductance	umhos	695	---	730	665	---	715	745	590	800
	Chloride	mg/l	43	---	47	33	---	40	49	52	57
	Turbidity	NTU	3	---	3	4.0	---	3.5	2.4	---	1.9
III-S	Soluble Organic Carbon	mg/l	9	13	13	16	13	15	16	8	10
	Soluble Kjeldahl Nitrogen	mg/l	0.79	0.82	0.95	0.81	0.98	1.0	0.82	1.0	1.5
	Soluble Phosphorus	mg/l	0.07	0.07	0.07	0.03	0.04	0.07	0.03	0.12	0.05
III-T	TOC	mg/l	14	17	16	13	9	12	16	12	12
	Total Phosphorus	mg/l	0.09	0.07	0.05	0.09	0.08	0.11	0.10	0.12	0.17
	Ammonia Nitrogen	mg/l	0.08	0.20	0.24	0.13	0.18	0.18	0.43	0.45	0.99
	Total Kjeldahl Nitrogen	mg/l	1.0	0.90	1.0	0.92	1.1	1.3	0.80	1.0	1.7
	NO_3+NO_2-N	mg/l	1.0	0.90	1.2	1.4	2.2	2.8	1.1	1.6	1.6
	Grease and Oil	mg/l	2	1	3	1	2	1	1	3	2

C = Creyts
L = Logan
W = Webster

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	-----November. 1980---- C	L	W	----December. 1980----- C	L	W	-----January, 1981----- C	L	W
IV	BOD	mg/l	2	---	3	1.7	---	2	2.6	<1	3.5
V	COD	mg/l	25	---	30	33	---	32	22	57	24
VI	Lead	mg/l	<0.005	---	0.007	<0.005	---	<0.005	<0.005	0.001	0.005
	Iron	mg/l	0.32	---	0.4	---	---	---	---	0.55	---
	Zinc	mg/l	0.025	---	0.03	0.016	---	0.016	0.048	0.06	0.02
	Chromium	mg/l	<0.005	---	<0.005	<0.005	---	<0.005	0.014	0.019	0.01
	Copper	mg/l	0.005	---	0.011	0.006	---	0.010	0.006	0.005	0.01
	Nickel	mg/l	0.013	---	0.015	0.008	---	0.012	0.011	0.004	0.014
	Cadmium	mg/l	<0.002	---	<0.002	<0.002	---	<0.002	0.002	0.0002	<0.002
VIII	Total Flouride	mg/l	---	---	---	0.19	0.20	0.20	0.32	---	0.42
IX	Orthophosphate	mg/l	0.043	---	0.091	0.031	---	0.065	0.077	---	0.098
X	Phenolics	mg/l	0.0006	---	0.0006	<0.0005	---	<0.0005	<0.0005	---	0.0006
XI	Total Sulfide	mg/l	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	---	<0.01
	Flow Rate	cfs	242	---	525	684	---	934	280	---	453

C = Creyts
L = Logan
W = Webster

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TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	----February, 1981-----			---- --March, 1981-------			-----April, 1981-------		
			C	L	W	C	L	W	C	L	W
I	Total Solids	mg/1	456	580	546	690	440	460	394	450	439
	Total Suspended Solids	mg/1	<2	5	6	5	4	5	7	26	10
	pH	s.u.	7.70	7.95	7.90	7.80	7.98	7.80	8.30	8.15	8.40
	Total Alkalinity	mg/1	260	260	240	---	190	---	215	230	230
	Specific Conductance	umhos	780	770	955	515	520	585	595	600	660
	Chloride	mg/1	47	860	106	25	27	30	32	33	38
	Turbidity	NTU	3.1	4	3.1	2.2	2	2.5	2.1	4	2.4
III-S	Soluble Organic Carbon	mg/1	10	8	7	12	15	14	12	16	14
	Soluble Kjeldahl. Nitrogen	mg/1	0.94	1.4	1.2	0.88	0.94	1.0	0.73	0.82	0.85
	Soluble Phosphorus	mg/1	0.04	0.05	0.05	0.03	0.05	0.04	0.06	0.06	0.05
III-T	TOC	mg/1	14	9	7	16	19	16	12	13	12
	Total Phosphorus	mg/1	0.09	0.11	0.09	0.05	0.08	0.07	0.08	0.08	0.11
	Ammonia Nitrogen	mg/1	0.45	0.52	0.48	0.14	0.26	0.27	0.12	0.01	0.01
	Total Kjeldahl Nitrogen	mg/1	1.2	1.2	1.2	0.89	1.1	1.0	0.82	1.0	0.92
	NO_3+NO_2-N	mg/1	1.1	1.7	2.2	1.6	2.1	2.2	0.62	1.1	1.3
	Grease and Oil	mg/1	1.0	1	2	3	1	3	1	2	2

C = Creyts
L = Logan
W = Webster

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	----February, 1981----- C	L	W	------March, 1981------ C	L	W	------April, 1981------ C	L	W
IV	BOD	mg/l	1.1	<1	1.6	1.9	<1	2.1	2.2	2	3.1
V	COD	mg/l	29	23	26	49	40	47	24	22	30
VI	Lead	mg/l	0.005	0.001	0.004	---	0.001	---	<0.001	<0.001	<0.001
	Iron	mg/l	0.96	0.48	0.37	---	0.29	---	0.45	0.42	0.43
	Zinc	mg/l	0.08	0.05	0.05	---	0.05	---	0.04	0.04	0.04
	Chromium	mg/l	0.014	0.012	0.003	---	0.011	---	0.003	0.002	0.003
	Copper	mg/l	0.006	0.01	0.008	---	0.005	---	<0.01	<0.01	0.01
	Nickel	mg/l	0.010	0.008	0.015	---	0.003	---	0.007	0.006	0.010
	Cadmium	mg/l	0.0013	<0.0002	<0.0002	---	<0.0002	---	<0.0002	<0.0002	<0.0002
VIII	Total Flouride	mg/l	---	0.16	---	0.16	0.16	0.18	0.19	0.32	0.36
IX	Orthophosphate	mg/l	0.048	0.03	0.061	0.028	0.03	0.042	0.015	---	0.053
X	Phenolics	mg/l	<0.0005	---	0.0006	0.0022	0.002	0.0034	<0.0005	0.006	<0.0005
XI	Total Sulfide	mg/l	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
	Flow Rate	cfs	300	---	517	1,280	---	2,040	625	---	1,010

C = Creyts
L = Logan
W = Webster

** Data from U.S. Geological Survey.

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	May, 1981			June, 1981			July, 1981		
			C	L	W	C	L	W	C	L	W
I	Total Solids	mg/l	391	---	437	410	450	450	480	500	500
	Total Suspended Solids	mg/l	11	---	15	7	14	14	21	14	17
	pH	s.u.	8.10	---	8.10	7.85	8.03	7.99	8.29	8.05	8.14
	Total Alkalinity	mg/l	225	---	230	220	230	230	250	250	170
	Specific Conductance	umhos	585	---	650	610	610	650	620	640	680
	Chloride	mg/l	28	---	35	39	36	44	39	41	49
	Turbidity	NTU	3.3	---	4.7	4	5	4	14	7	7
III-S	Soluble Organic Carbon	mg/l	16	18	18	16	13	14	---	---	---
	Soluble Kjeldahl Nitrogen	mg/l	0.90	1.1	1.1	0.87	0.94	0.94	---	---	---
	Soluble Phosphorus	mg/l	0.03	0.04	0.05	0.12	0.09	0.10	---	---	---
III-T	TOC	mg/l	22	20	22	17	16	18	5	10	12
	Total Phosphorus	mg/l	0.06	0.09	0.10	0.16	0.15	0.16	0.11	0.17	0.22
	Ammonia Nitrogen	mg/l	0.10	0.09	0.32	0.10	0.14	0.12	<0.01	0.12	<0.01
	Total Kjeldahl Nitrogen	mg/l	1.1	1.0	1.4	1.6	0.97	1.1	1.4	1.2	1.2
	NO_3+NO_2-N	mg/l	0.49	0.93	0.96	0.54	0.94	1.2	0.45	1.1	1.6
	Grease and Oil	mg/l	2	2	3	1	1	1	<1	2	1

C = Creyts
L = Logan
W = Webster

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	May, 1981 C	May, 1981 L	May, 1981 W	June, 1981 C	June, 1981 L	June, 1981 W	July, 1981 C	July, 1981 L	July, 1981 W
IV	BOD	mg/1	2.4	---	3	---	4	---	5	2	2
V	COD	mg/1	30	---	30	---	34	---	30	27	20
VI	Lead	mg/1	<0.001	0.002	0.002	0.002	0.003	0.004	---	---	---
	Iron	mg/1	0.40	0.52	0.50	0.46	0.37	0.42	---	---	---
	Zinc	mg/1	0.05	0.04	0.06	0.05	0.05	0.05	---	---	---
	Chromium	mg/1	0.003	0.001	<0.001	0.004	0.003	0.003	---	---	---
	Copper	mg/1	<0.01	<0.01	<0.01	<0.01	<0.01	0.01	---	---	---
	Nickel	mg/1	0.006	0.005	0.014	0.016	0.017	0.026	---	---	---
	Cadmium	mg/1	<0.0002	0.0002	0.0002	---	---	---	---	---	---
VIII	Total Flouride	mg/1	0.18	0.19	0.20	---	---	---	---	---	---
IX	Orthophosphate	mg/1	0.019	---	0.061	---	---	---	---	---	---
X	Phenolics	mg/1	<0.0005	---	<0.0005	---	---	---	---	---	---
XI	Total Sulfide	mg/1	<0.01	<0.01	<0.01	---	---	---	---	---	---
	Flow Rate	cfs	843	---	1,290	627	---	683	274	---	361

C = Creyts
L = Logan
W = Webster

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	---August, 1981--- C	L	W	---September, 1981--- C	L	W	---October, 1981--- C	L	W	---November, 1981--- C	L	W
IV	BOD	mg/l	2	2	2	<1	2	2	1	<1	1	<1	<1	<1
V	COD	mg/l	30	32	33	24	24	29	64	40	36	31	35	31
VI	Lead	mg/l	<0.001	0.008	0.007	---	---	---	0.004	0.006	0.005	---	---	---
	Iron	mg/l	0.53	0.98	1.2	---	---	---	0.88	1.3	1.4	---	---	---
	Zinc	mg/l	0.09	0.10	0.10	---	---	---	0.10	0.10	0.10	---	---	---
	Chromium	mg/l	0.004	0.004	0.024	---	---	---	0.008	0.006	0.010	---	---	---
	Copper	mg/l	<0.01	0.01	0.01	---	---	---	0.01	0.01	<0.01	---	---	---
	Nickel	mg/l	0.013	0.012	0.051	---	---	---	0.009	0.005	0.007	---	---	---
	Cadmium	mg/l	---	---	---	---	---	---	---	---	---	---	---	---
VIII	Total Flouride	mg/l	---	---	---	---	---	---	---	---	---	---	---	---
IX	Orthophosphate	mg/l	---	---	---	---	---	---	---	---	---	---	---	---
X	Phenolics	mg/l	---	---	---	---	---	---	---	---	---	---	---	---
XI	Total Sulfide	mg/l	---	---	---	---	---	---	---	---	---	---	---	---
	Flow Rate	mg/l	536	---	646	404	---	592	961**	---	2,860**	---	---	1,010**

C = Creyts
L = Logan
W = Webster

** Data from U.S. Geological Survey.

TABLE VII-15 (Continued)

GRAND RIVER WATER QUALITY DATA

GROUP	PARAMETER	UNITS	---August 1981---			---September 1981---			---October 1981---			---November 1981---		
			C	L	W	C	L	W	C	L	W	C	L	W
I	Total Solids	mg/l	400	400	430	430	480	510	410	400	400	450	480	480
	Total Suspended Solids	mg/l	13	28	38	13	12	32	12	24	32	7	12	11
	pH	s.u.	7.73	7.85	7.86	7.82	7.77	7.91	7.70	7.58	7.58	7.92	7.90	7.94
	Total Alkalinity	mg/l	210	200	190	180	180	180	190	160	150	230	240	240
	Specific Conductance	umhos	540	540	550	500	610	610	520	480	470	650	690	690
	Chloride	mg/l	33	30	36	38	46	49	26	22	2	31	34	36
	Turbidity	NTU	4	14	16	6	8	16	7	17	17	4	6	5
III-S	Soluble Organic Carbon	mg/l	---	---	---	---	---	---	---	---	---	---	---	---
	Soluble Kjeldahl Nitrogen	mg/l	---	---	---	---	---	---	---	---	---	---	---	---
	Soluble Phosphorus	mg/l	---	---	---	---	---	---	---	---	---	---	---	---
III-T	TOC	mg/l	12	13	10	9	10	10	14	12	16	13	14	13
	Total Phosphorus	mg/l	0.21	0.10	0.21	0.10	0.13	0.18	0.10	0.13	0.16	0.07	0.06	0.13
	Ammonia Nitrogen	mg/l	0.08	0.08	0.05	0.11	0.08	0.06	0.05	0.07	0.09	0.04	0.16	0.14
	Total Kjeldahl Nitrogen	mg/l	0.92	1.0	1.8	0.84	0.76	1.0	1.5	0.95	0.98	0.78	1.1	0.88
	NO_3+NO_2-N	mg/l	0.47	0.51	1.0	0.63	1.2	1.6	1.3	3.3	3.7	0.90	1.5	1.6
	Grease and Oil	mg/l	<1	1	2	---	---	---	2	2	2	1	2	1

C = Creyts
L = Logan
W = Webster

TABLE VII-16

Comparison of ENCOTEC and MDNR
Grand River Data

GROUP ID	PARAMETER	UNITS	November 1979 CREYTS ENCOTEC	November 1979 CREYTS MDNR	November 1979 WEBSTER ENCOTEC	November 1979 WEBSTER MDNR	December 1979 CREYTS ENCOTEC	December 1979 CREYTS MDNR	December 1979 WEBSTER ENCOTEC	December 1979 WEBSTER MDNR
I	Total Solids	mg/l	440	445	430	493	450	446	490	487
	Total Suspended Solids	mg/l	3	1	16	9	1	2	1	3
	pH	s.u.	8.03	7.90	7.93	7.70	8.01	8.00	8.05	8.10
	Total Alkalinity	mg/l	223	225	201	210	220	225	200	215
	Specific Conductance	umhos	---	---	---	---	660	670	710	725
	Chloride	mg/l	57	55	66	65	55	47	73	57
	Turbidity	NTU/FTU	2.2	2.1	6.2	3.2	5.4	1.3	3.2	1.2
IV	BOD	mg/l	<1	1.8	4	2.8	1.0	1.7	1.0	1.5
V	COD	mg/l	20	18	12	34	25	26	20	27
VI	Lead	mg/l	<0.001	<0.005	0.002	<0.005	<0.001	<0.005	<0.001	<0.005
	Iron	mg/l	0.18	0.26	0.20	0.39	0.16	0.20	0.14	0.19
	Zinc	mg/l	0.027	0.015	0.037	0.025	0.043	0.025	0.048	0.027
	Chromium	mg/l	0.009	0.003	0.013	0.004	0.017	<0.010	0.026	<0.010
	Copper	mg/l	<0.001	0.004	0.007	0.012	<0.001	0.004	0.009	0.009
	Nickel	mg/l	0.04	0.04	0.035	0.030	0.030	0.035	0.034	0.026
	Cadmium	mg/l	<0.0001	<0.002	<0.0001	<0.002	<0.0001	<0.002	0.0001	0.002
IX	Orthophosphate	mg/l	0.08	0.091	0.18	0.112	0.06	0.068	0.15	0.121
X	Phenolics	mg/l	0.006	0.003	0.005	0.005	0.004	0.004	0.008	0.005

TABLE VII-16(Continued)

Comparison of ENCOTEC AND MDNR
Grand River Data

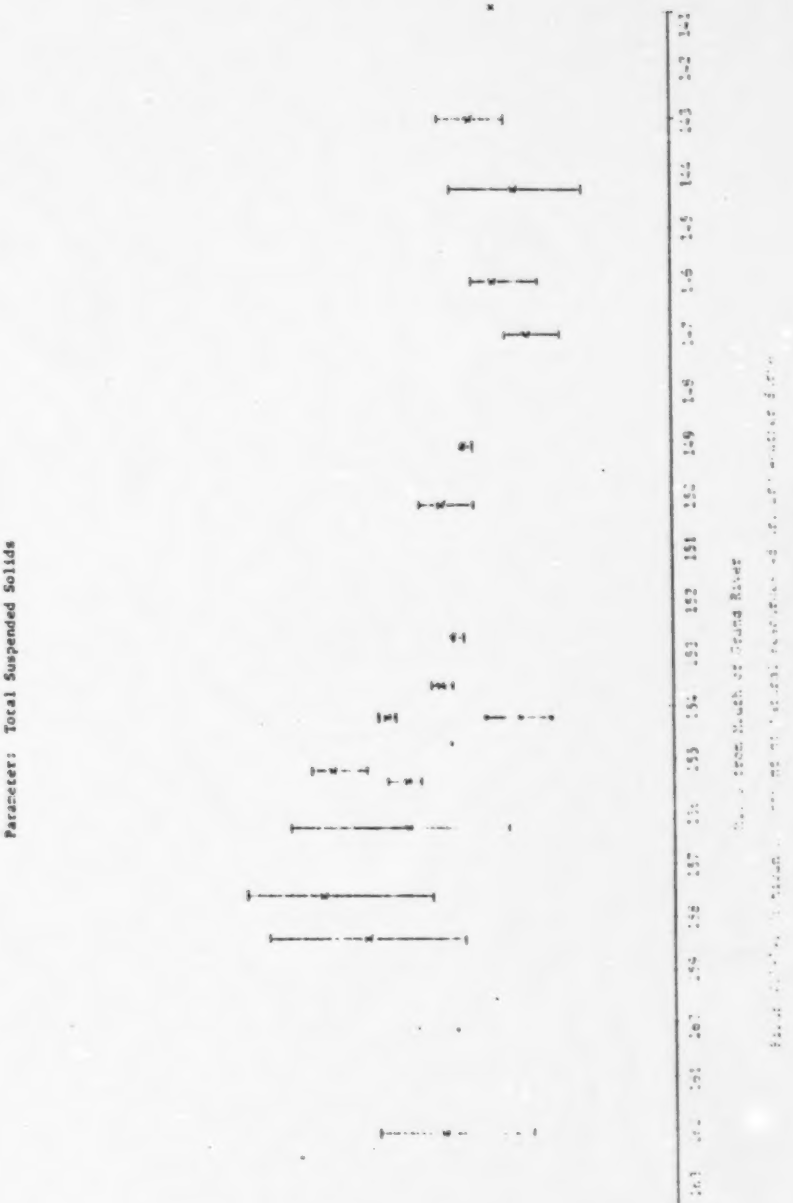
GROUP ID	PARAMETER	UNITS	-------January 1980-----				-------February 1980-----			
			CREYTS		WEBSTER		CREYTS		WEBSTER	
			ENCOTEC	MDNR	ENCOTEC	MDNR	ENCOTEC	MDNR	ENCOTEC	MDNR
I	Total Solids	mg/1	480	435	510	475	380	346	410	396
	Total Suspended Solids	mg/1	4	7	4	3	11	6	15	12
	pH	s.u.	7.75	7.80	7.88	7.80	7.84	7.80	7.80	7.70
	Total Alkalinity	mg/1	190	200	210	215	160	159	160	155
	Specific Conductance	umhos	670	675	670	740	480	550	520	590
	Chloride	mg/1	55	47	60	49	48	46	53	40
	Turbidity	NTU	4.2	2.6	3.2	2.5	7.0	7.1	12	15
IV	BOD	mg/1	1.0	1.8	<1	1.8	4.0	4.8	3.0	6.2
V	COD	mg/1	20	26	27	28	32	32	31	31
VI	Lead	mg/1	<0.001	<0.005	<0.001	<0.005	0.001	0.008	0.001	0.018
	Iron	mg/1	0.20	0.38	0.15	0.32	0.43	0.70	0.52	0.87
	Zinc	mg/1	0.063	0.016	0.043	0.015	0.05	0.035	0.04	0.030
	Chromium	mg/1	0.017	<0.005	0.015	<0.005	0.023	0.006	0.022	0.006
	Copper	mg/1	<0.001	0.004	0.005	0.007	0.004	0.006	0.006	0.009
	Nickel	mg/1	0.017	0.018	0.024	0.023	0.022	0.016	0.030	0.005
	Cadmium	mg/1	0.0001	<0.002	<0.0001	<0.002	0.0002	<0.002	<0.0001	<0.002
IX	Orthophosphate	mg/1	0.04	0.059	0.11	0.106	---	0.069	---	0.142
X	Phenolics	mg/1	0.004	0.003	0.005	0.002	---	0.011	---	0.019

TABLE VII-16 (Continued)

Comparison of ENCOTEC AND MDNR
Grand River Data

GROUP ID	PARAMETER	UNITS	-----April 1980---- CREYTS ENCOTEC	CREYTS MDNR	WEBSTER ENCOTEC	WEBSTER MDNR	------May 1980------ CREYTS ENCOTEC	CREYTS MDNR	WEBSTER ENCOTEC	WEBSTER MDNR
I	Total Solids	mg/l	530	433	550	482	420	469	470	522
	Total Suspended Solids	mg/l	14	25	22	26	6	13	22	42
	pH	s.u.	8.17	8.30	8.24	8.20	8.19	8.50	8.15	8.20
	Total Alkalinity	mg/l	180	165	190	191	220	210	210	196
	Specific Conductance	umhos	570	535	550	595	630	606	680	646
	Chloride	mg/l	37	28	38	36	38	37	52	46
	Turbidity	NTU	5.0	4.9	5.0	5.5	5.0	3.3	6.0	6.2
IV	BOD	mg/l	---	2.7	---	4.5	2.0	2.8	5.0	9.0
V	COD	mg/l	46	33.0	49	35.0	35	33.0	44	51.0
VI	Lead	mg/l	0.001	<0.005	0.002	0.006	<0.001	<0.005	0.004	0.010
	Iron	mg/l	0.37	0.82	0.54	0.71	0.45	0.55	0.68	1.0
	Zinc	mg/l	0.033	0.020	0.035	0.030	0.030	0.030	0.050	0.050
	Chromium	mg/l	0.016	0.005	0.013	0.005	0.031	<0.005	0.015	0.006
	Copper	mg/l	0.001	0.006	0.007	0.010	0.001	0.022	0.008	0.013
	Nickel	mg/l	0.016	0.016	0.018	0.016	0.016	0.018	0.016	0.020
	Cadmium	mg/l	0.0002	<0.002	0.0011	<0.002	0.0001	<0.002	0.0002	<0.002
IX	Orthophosphate	mg/l	---	0.008	---	0.041	---	0.033	---	0.048
X	Phenolics	mg/l	---	0.004	---	0.004	---	0.002	---	0.003

Parameter: Total Suspended Solids



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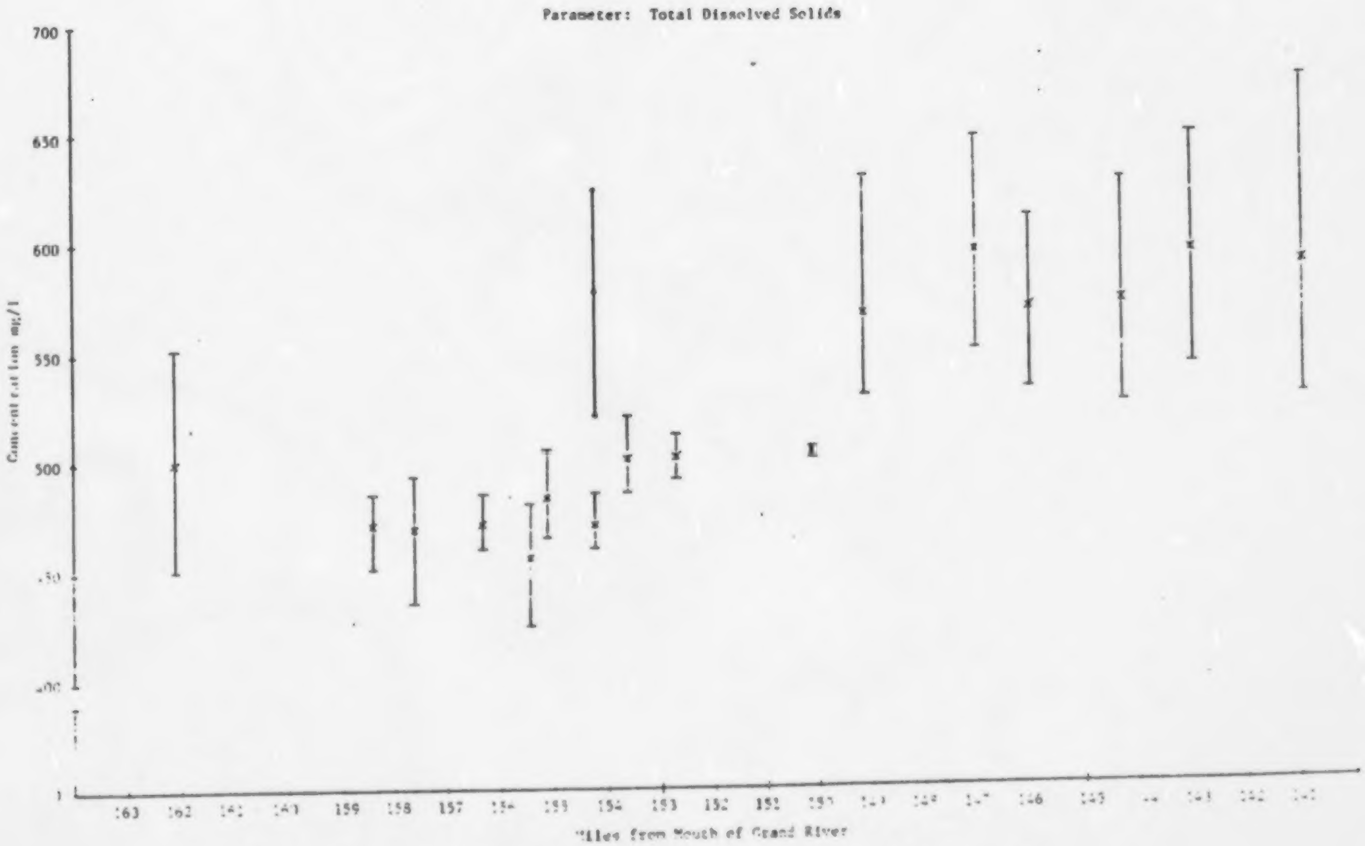

Figure VII-15. Michigan Department of Natural Resources [illegible]

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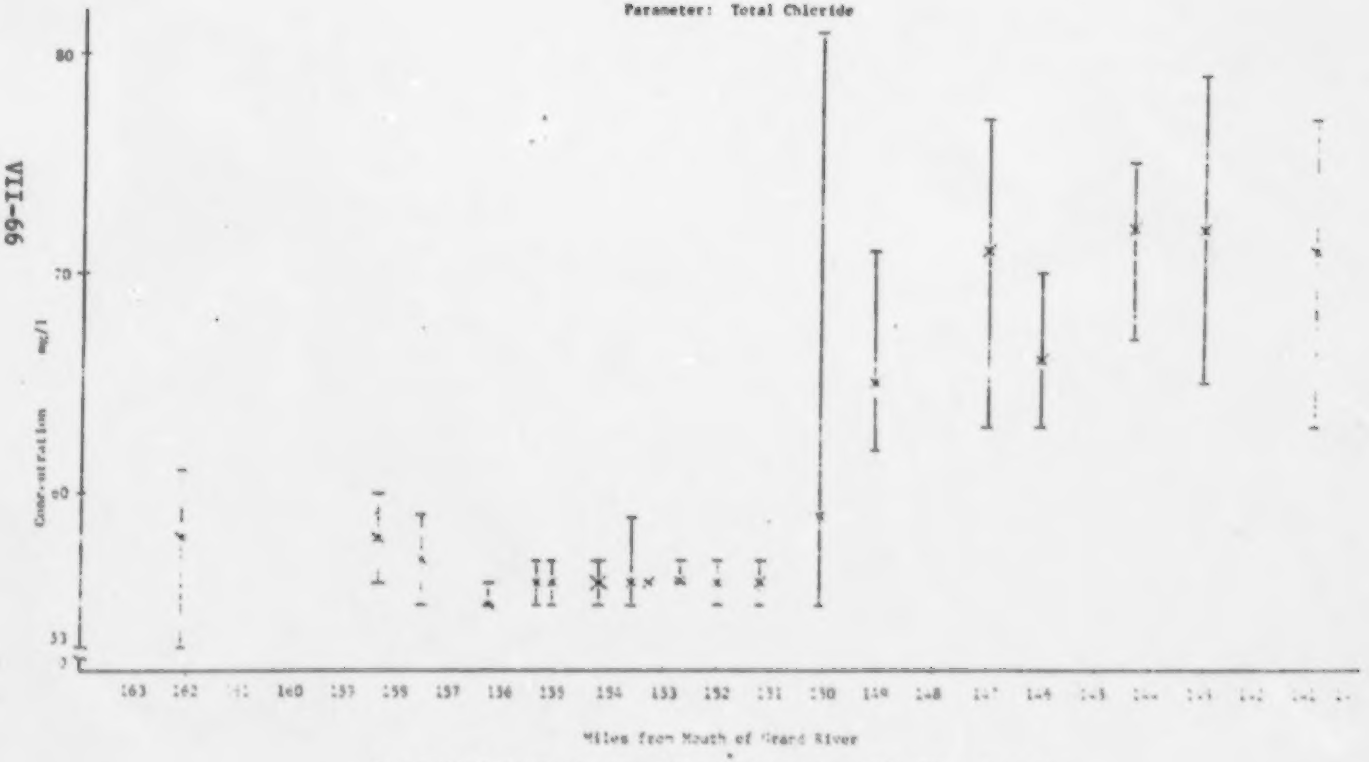

Figure VII-[illegible] Michigan Department of Natural Resources [illegible] Survey

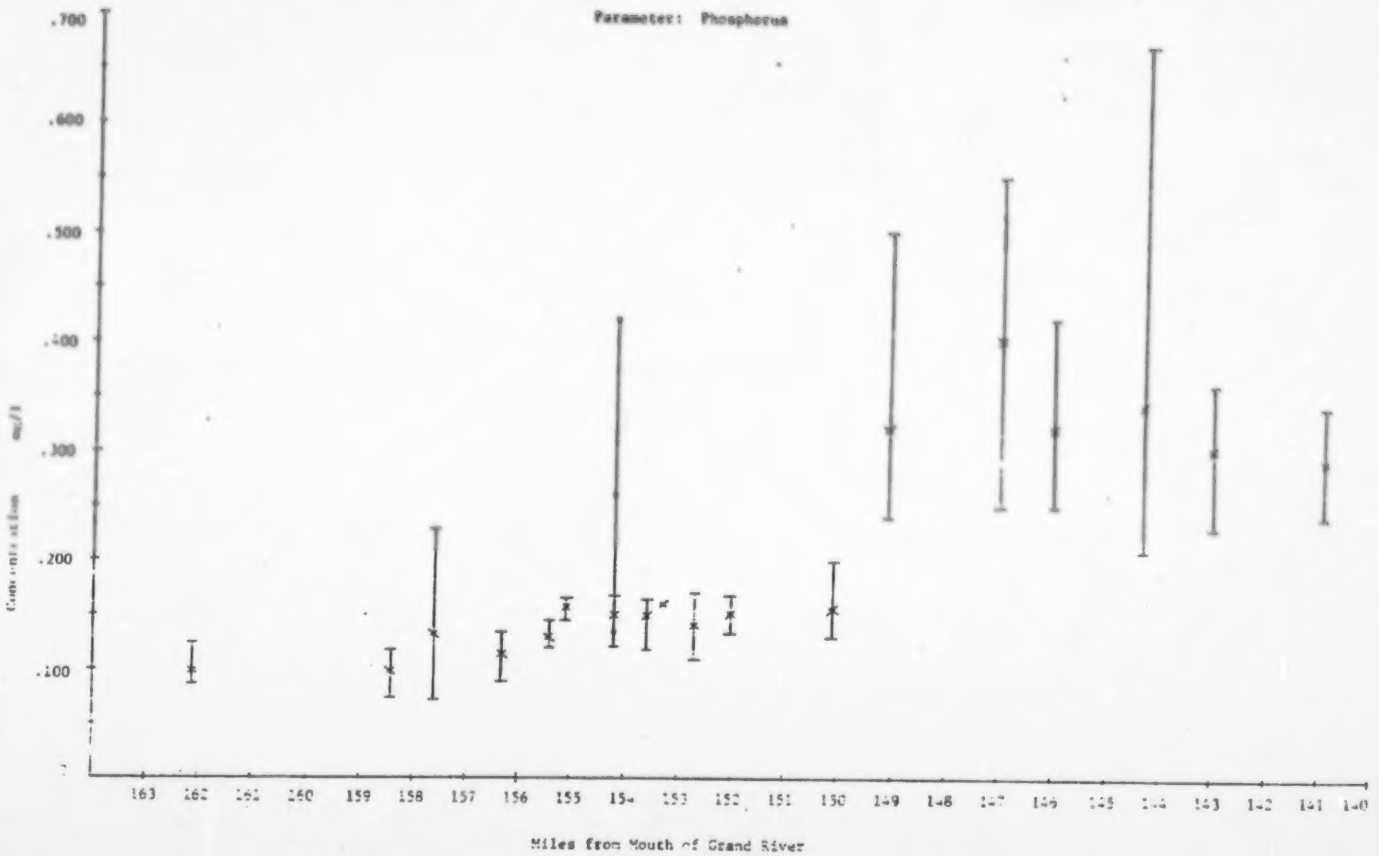

Figure VII-17. Michigan Department of Natural Resources [illegible] Wet Weather Survey

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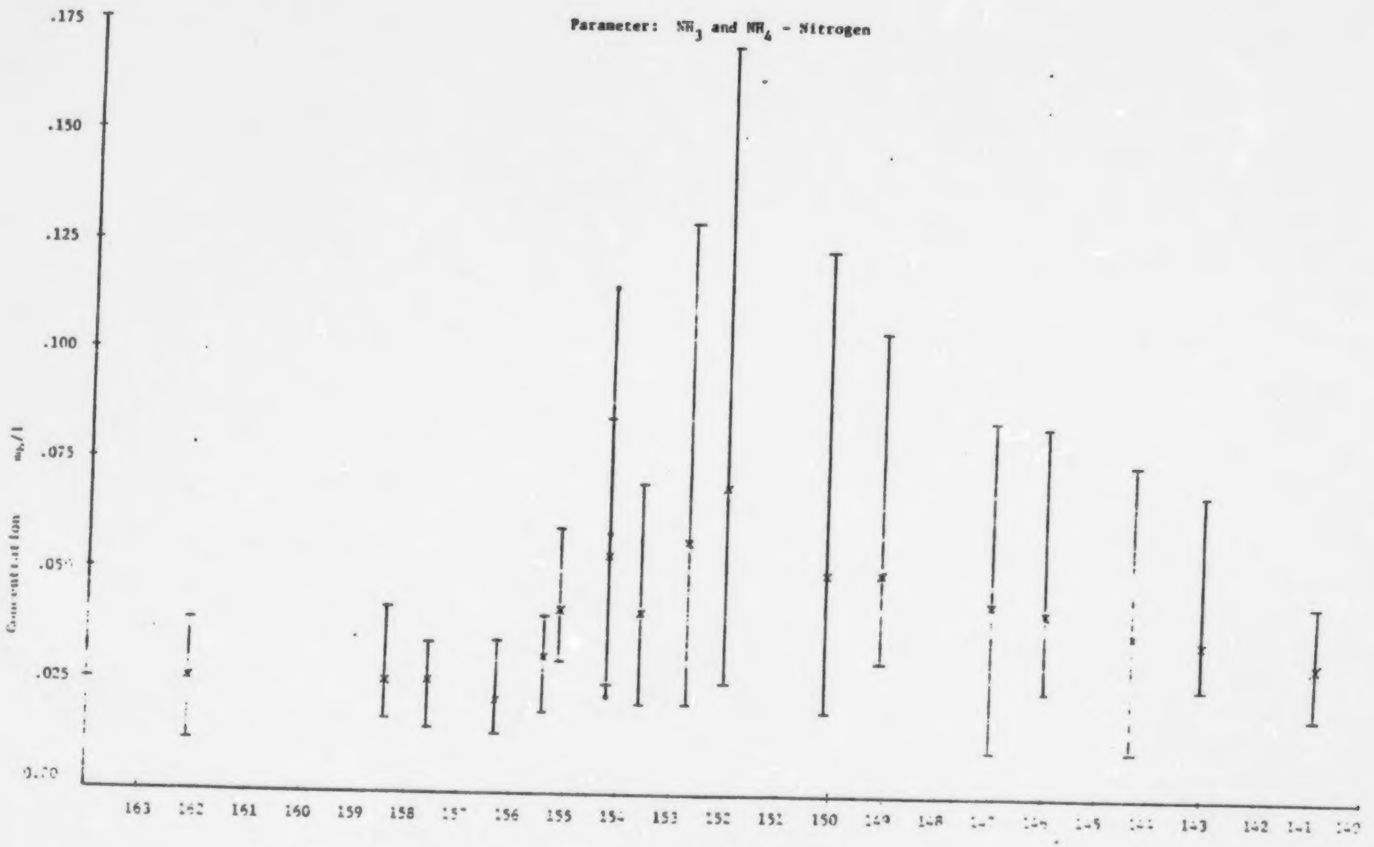

Miles from Mouth of Grand River

Figure VII-[illegible] Michigan Department of Natural Resources 48 hr. Dry Weather Survey

Parameter: NO_3 - Nitrogen

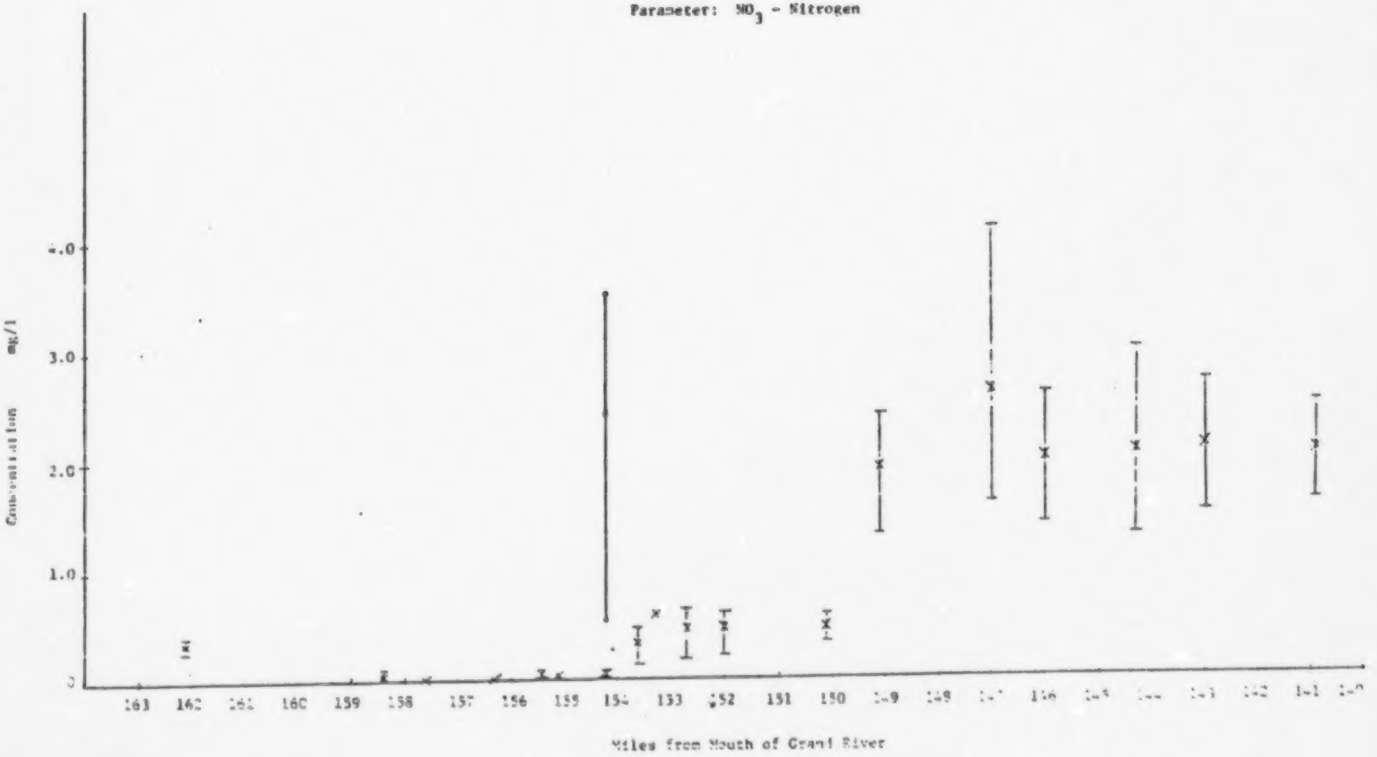

Figure VII-69. Michigan Department of Natural Resources [illegible]

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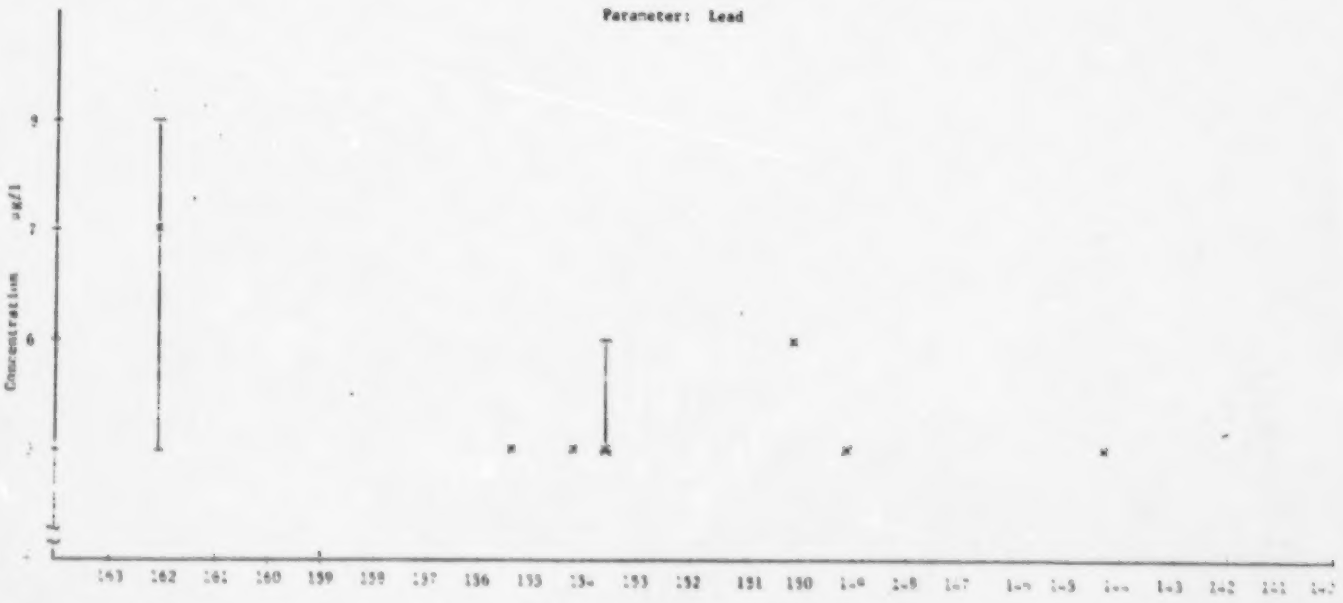

Figure [illegible] Michigan Department of Natural Resources 48 Hr. Dry Weather Survey

Violations of the 25 mg/l suspended solids guideline are seen at several of the upstream stations, and possible violations of the 500 mg/l total dissolved solids standard are seen at several of the downstream stations. Phosphorus concentrations in excess of the 0.5 mg/l guideline occur downstream of the Lansing wastewater treatment plant.

Additional water quality data for the Lansing area rivers is presented in the regional 208 reports and the Lansing CSO reports. Water quality surveys and river modeling data are reported or referenced in these publications.[7,9]

DESCRIPTION OF WATER QUALITY PROBLEMS

Introduction

There are three main drainage basins, as described earlier, which either impact or flow through the Lansing urban area. Two of these basins, Sycamore Creek and the Red Cedar River, join the Grand River in Lansing. Thus, in addressing the water quality problems and impacts for the Grand River, all three water resources must be considered. Therefore, during the description and discussion of water quality in the Grand River, all three basins will be included.

The 208 water quality management plan of 1977 was based on an assessment of the water quality conditions in the river basin as determined from existing water quality data as well as data generated during the 208 study, using mathematical modeling techniques. Much of the information used has been included in previous parts of this section (water quality standards, flow, water quality data, sources). Additional investigations on the three water courses have been accomplished since 1977, including the river water quality data collected during this study. All of this additional work has substantiated the findings of the original 208 plan. The data collected during this study and other 208 follow-on programs have shown that water quality conditions have not changed substantially in the past five years. Therefore, the water quality problems described in the earlier 208 work (actual data and modeling) have been used as the basis of the description and assessment discussions presented in this report.[1,2,8]

The description of water quality conditions are referenced by comparisons to the State of Michigan water quality standards. However, as previously stated, numerical standards are only set for a very few parameters. Therefore, water quality guidelines have also been used for reference. While these guidelines do not have the "force of law", they serve as a basis for evaluating the general conditions in the river and provide a basis for targeting water quality problems, assessing sources, and assessing potential control alternatives.

Grand River

The Grand River currently experiences violations of the dissolved oxygen (DO) standards in both the urban area and in downstream segments of the river. DO levels below 4.0 mg/l have been measured during both dry and wet weather conditions at many locations throughout the urban and downstream areas of the river. The low dissolved oxygen condition is the most significant water quality problem in the river. The dissolved oxygen violations are projected to continue in the river even with the upgrading of the wastewater treatment plants to tertiary treatment levels. Low flow dissolved oxygen levels can be directly attributed to significant benthic oxygen demand in many parts of the river in the Lansing area. High flow dissolved oxygen problems have been measured and are directly attributable to the comined sewer overflows discharging to the river.

The dissolved oxygen problem is the only numerical violation of state water quality standards as they currently exist. However, high levels of suspended solids, phosphorus, and fecal coliform are typically present in the river and exceed the water quality guidelines. These high concentrations typically occur during both low and high flow conditions.

During average or median flow conditions, the Grand River water quality could be considered fair and typical of many rivers passing through urban areas. The summarized data presented in Figures VII-2 through 13 show typical water quality concentrations of:

DO - 6-8 mg/l

TSS - 10-20 mg/l
T-P - 0.2-0.4 mg/l
TKN - 0.8-1.2 mg/l
TDS - 450-500 mg/l

These levels are generally within the water quality guidelines for river systems. Heavy metals are generaly low and well below toxic levels for fish species and below drinking water guidelines.

Further inspection of the water quality data, however, shows that wide fluctuations occur in water quality conditions and very high levels of pollutants are often present in the river. This is particularly true for suspended solids and phosphorus. Suspended solids levels of 40 to 80 mg/l are not uncommon during and following runoff periods, and phosphorus levels of 0.5 to 1 mg/l have been observed during both dry and wet weather periods. These concentration levels clearly present objectionable water quality conditions and impair the use of the river for fishing, swimming and related recreational uses. Thus, in addition to the dissolved oxygen problem, suspended solids and phosphorus are considered water quality problems in the Grand River in the Lansing urban area and downstream segments. The sources of these pollutant loads are clearly associated with the urban environment as will be discussed in the next section of this report.

Red Cedar River

The Red Cedar River is a main tributary to the Grand River at Lansing and is very similar to the Grand River in characteristics. Dissolved oxygen violations occur in the Red Cedar at each urban location along the river. DO levels approaching 0.0 mg/l have been measured in the past and levels below 4.0 mg/l occur frequently in the last few miles of stream in the urban area. Low DO levels occur during both dry and wet weather conditions. Dissolved oxygen problems have continued in some portions of the river even after the upgrading of the East Lansing wastewater treatmeant plant to a tertiary facility. The low flow DO violations are attributable to benthic oxygen demand as shown by recent studies of the river. Wet weather violations, as measured, are attributable to combined sewer overflows along the river. These low dissolved oxygen levels, in addition to impacting water quality in the river itself, are also an important factor in the Grand River. The Red Cedar represents approximately 25% of the flow in the Grand River as it passes through the Lansing area below the confluence of the two rivers. This input of low DO water to the Grand River immediately impacts the water quality conditions by substantially lowering the overall DO levels.

As in the case of the Grand River, the DO violations in the Red Cedar are the only typical violations of numerical water quality standards. The Red Cedar however, also exhibits wide variation in suspended solids, phosphorus, and nitrogen levels, primarily during low and high flow periods. Concentrations of suspended solids in the range of 40-80 mg/l and phosphorus levels of 0.2 to 1.0 mg/l are not uncommon, particularly during wet weather periods. These levels exceed water quality guidelines and create objectionable conditions in the river. The river passes through the Michigan State University campus and is considered an important recreational resource. However, following wet weather periods, the high levels of suspended solids impair the aesthetic conditions of the area and impede full use of the river.

As in the case of dissolved oxygen, high levels of pollutants in the Red Cedar have an immediate impact of the water quality of the Grand River from the confluence of the rivers downstream through the urban area. Any improvement in Grand River water quality must include improvement of the Red Cedar due to its large flow input to the Grand River.

Sycamore Creek

Sycamore Creek is a major tributary to the Red Cedar River representing one third of the area drained by the Red Cedar. The creek enters the river just upstream of the confluence of the Red Cedar and the Grand River. This creek is somewhat of a contrast to the two rivers. in that water quality standards for DO are not violated during either dry or wet weather periods. There is much less urban development in this basin in relationship to the flow in the creek. Under low flow conditions phosphorus levels will be elevated due to the Mason wastewater treatment plant but dissolved oxygen and suspended solids are not projected to be a problem. During wet weather, elevated levels of suspended solids and phosphorus occur in the creek and enter the combined river systems.

Impact of the Urban Area on Water Quality

The large amount of water quality data collected during the past twenty years clearly demonstrate the decrease in water quality of the Red Cedar and Grand Rivers as they pass through the urban area. Increased levels of solids, oxygen demanding materials, and nutrients occur as the rivers flow through the area. Dissolved oxygen levels decrease and biological conditions in the rivers deteriorate in response to the water quality conditions.

In order to determine control strategies to stop deterioration in water quality, determination of the sources of the pollutants must be made. Several studies and evaluations have been performed prior to this investigation including State of Michigan studies, 201 facility assessments, and finally the 208 program. The major sources addressed by these projects were municipal and industrial point source discharges, combined sewer overflows, and urban separate storm sewer discharges. Each of the above sources were found to contribute to water quality problems in the rivers under varying circumstances and to varying degrees. The results of these investigations, culminating in the 208 water quality plan in 1977, and subsequent work of which this project is a part, have shown one main factor which results in this significant water quality degradation. This factor is that in this specific area there is a large population contributing relatively large sewage and wet weather runoff flows to comparably small rivers. In the case of dry weather conditions, the wastewater treatment plants can account for 25 to 50 percent of the total river flow. During wet weather conditions, urban stormwater runoff (CSO and separate) can frequently double or triple the river flow. This condition clearly shows that if the principal sources of flow to the river exceed the water guidelines or standards for the river, then violations will occur in the river.[2]

The impact of point source discharges was addressed several years ago. A clear relationship between the wastewater discharges and low flow water quality conditions was established. As a result, the wastewater treatment plants have all been upgraded to tertiary treatment standards. Thus, the impact of these discharges on the rivers has been substantially reduced.

The 208 water quality program beginning in 1975 has been the principal source of information on the non-point source (CSO and separate systems) loadings and subsequent impacts on the Red Cedar and Grand Rivers. The original loading informaion developed in 1977 presented a summary of the various sources impacting these rivers. Subsequent studies on the combined sewer overflows substantiated the earlier work. In the source section of the current evaluation, additional data on the separate storm sewer systems has been presented. As stated in Section VI, the current data is comparable with the earlier results, and supports the importance of urban runoff to the water quality of these rivers.[8]

Table VII-17 shows the annual loading, dry weather loading, and wet weather loading for the pollutant sources in the river basins. This table is taken from the water quality management plan report. Using this data in conjunction with the water quality data for the rivers the importance of the various sources can be addressed.[1]

Both the Grand River and Red Cedar rivers have violations of the dissolved oxygen standards. The principal cause of violations during wet weather is the combined sewer overflows and non-point source loadings along the river. In addition, the organic solids discharged from combined sewer overflows tend to deposit along the bottom of the river resulting in a benthic oxygen demand. The impact of urban runoff in the combined sewer areas is quite clear. Oxygen depletion is occurring at many points along the river and is the principal source of DO water quality standard violations.

Urban runoff is also a primary source of elevated suspended solids and phosphorus concentrations in these

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TABLE VII-17

Sycamore Creek

7 Day, 10 Year Low Flow

	BOD				TKN				TSS				TP			
	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS
Pound/Day	92	14	0	70	18	2	0	16	92	23	0	292	9	1	0	2.1
Percent	52	8		40	50	6		44	23	6		71	74	8		18

10 Year, 2 Hour Summer Storm

	BOD				TKN				TSS				TP			
	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS
Pound/Day	92	14	3,800	49,600	18	2	500	3,130	92	23	8,700	142,320	5	1	200	730
Percent	<1	<1	7	93	<1	<1	14	85	<1	<1	6	94	1	<1	21	78

Annual Load

	BOD				TKN				TSS				TP			
	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS
Pound/Year	33,600	5,100	51,700	674,600	6,600	700	6,800	42,500	33,600	8,400	118,300	1,935,400	3,300	400	2,700	9,980
Percent	4	<1	7	89	12	1	12	75	2	1	6	92	20	2	17	61

TABLE VII-17 (Continued)
Red Cedar River

7 Day, 10 Year Low Flow

	BOD				TKN				TSS				TP			
	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS
Pound/Day	1,047	14	0	242	233	2	0	66	1,070	23	0	1,456	103	1	0	11
Percent	80	1		19	78	<1		22	42	1		57	90	<1		10

10 Year, 2 Hour Summer Storm

	BOD				TKN				TSS				TP			
	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS
Pound/Day	1,047	14	94,600	118,630	233	2	8,400	8,660	92	23	106,600	352,840	99	1	4,200	1,902
Percent	<1	<1	44	56	1	<1	49	50	<1	<1	23	77	1	<1	68	31

Annual Load

	BOD				TKN				TSS				TP			
	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS
Pound/Year	382,200	5,100	1,286,600	1,688,600	84,900	8,400	114,300	132,500	390,900	8,400	1,449,800	4,798,500	37,760	400	57,100	25,880
Percent	11	<1	39	50	25	2	34	39	6	<1	22	72	31	<1	47	22

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TABLE VII-17 (Continued)

Grand River

7 Day, 10 Year Low Flow

	BOD				TKN				TSS				TP			
	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS
Pound/Day	4,574	76	0	1,628	1,084	7	0	414	4,700	1,580	0	5,731	423	18	0	80
Percent	73	1		26	72	<1		28	39	13		48	81	3		16

10 Year, 2 Hour Summer Storm

	BOD				TKN				TSS				TP			
	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS
Pound/Day	4,574	76	303,460	168,970	1,084	7	33,140	14,160	4,700	1,580	547,200	533,520	423	18	13,400	2,900
Percent	<1	<1	64	36	2	<1	68	30	<1	<1	50	50	3	<1	80	17

Annual Load

	BOD				TKN				TSS				TP			
	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS	Mun.	Ind.	CSO	NPS
Pound/Year	1,669,500	27,700	4,127,200	2,671,900	395,660	2,550	451,000	227,300	1,715,500	576,700	7,428,200	7,917,400	154,390	6,570	182,620	40,170
Percent	20	<1	49	31	37	<1	42	21	10	3	42	45	40	2	47	11

river systems. As stated in the description of current water conditons, high levels of both of these pollutants have been measured in the river systems. Both combined sewer overflows and non-point sources are the cause of the water quality deterioration. Control of urban runoff is the only alternative if improvement in general water quality is to be attained.

In summary, it is well documented that runoff from urban areas during rain events are resulting in water quality deterioration in these river systems. The main water quality problems identified for the river systems is dissolved oxygen depletion, high levels of suspended solids and enrichment of the waters with nutrients, particularly phosphorus. The combined sewer overflows have the largest impact on the dissolved oxygen levels and have the most immediate impact on the water quality in the urban area. Urban separate sewer systems and rural non-point sources contribute a large amount of suspended material resulting in very turbid waters which impair the recreational use of the rivers. Phosphorus is contributed by all of the sources and contributes to diurnal oxygen fluctuations through stimulation of biological activity. Finally, high levels of fecal coliform are present in the urban runoff. These inputs to the rivers result in high concentrations of fecal coliform in the rivers following storm events. Whether these high levels present a public health problem in respect to swimming is open to debate. However, given the current standards for fecal coliform as an indicator of unsafe conditions, the river would not be swimmable during these periods. The point is somewhat moot, however, in that the high turbidity due to suspended solids discharged during runoff periods results in a water quality aesthetically unpleasing and generally results in no one wishing to swim in the rivers during these periods.

Water Quality Management Plan - Control Alternatives

The water quality management plan prepared in 1977 by the Tri-County Regional Planning Commission addressed the problems described earlier. This plan included control of the combined sewer overflows, urban separate storm drains, and rural non-point sources. The plan was based on the available information and included a number of alternate control strategies. Part of the plan also included recommendations for additional evaluations of the combined sewer overflow loadings, urban separate system loadings and control alternatives. The combined sewer overflow studies were performed as referenced earlier. These studies supported the plan assumptions, demonstrating the impact of combined sewer overflows on the rivers. This current work addressed the confirmation of the pollutant loadings from separate systems and evaluated sedimentation basins and the upsize pipes for their applicability for runoff water quality control.

As stated previously, the loadings and river evaluation of this work supports the original water quality plan assumptions quite well. This study shows that the loadings used in the original evaluation are within reasonable limits. Therefore, the requirements for control of suspended solids and phosphorus in the urban runoff as determined in the 1977 report can be used to indicate the level of removal needed to meet water quality standards and guidelines.

The water quality management plan showed, in general, that removal of suspended solids and phosphorus were required in both CSO and separate systems. The details of the alternatives which were proposed to achieve this control are available in the 1977 plan report. In summary, on an area-wide basis, the removal requirements were shown as follows:

Suspended Solids - 25-40%
Total Phosphorus - 25-40%

Methods for attaining these removals varied, but it can be seen that overall reductions could be achieved by uniform low level removals in all runoff waters or high level removals of a portion of the runoff waters.

The results of this study show that the on-line retention basin is capable of controlling the two pollutants, suspended solids and phosphorus, to a high degree. This control strategy is applicable to the problems associated with the separate storm systems and this river system. In addition, this technology is readily applicable to developing areas which will only add to the existing water quality problems if they are permitted to develop without controls. It also appears that, given the high degree of control achieved by this type of basin, some urban runoff could be allowed to be uncontrolled and blended with the effluent of the retention basin to achieve the overall desired objective. This provides for a degree of flexibility in implementing control alternatives.

The upsized pipes were much less effective in controlling suspended solids and phosphorus. However the longer pipe did show removals in the range of 25% (concentration basis). A somewhat larger pipe, with an improved design to minimize scour, could possibly achieve 25-35% removal on a routine basis. These removals are lin the same range as those required in the overall regional area. Thus, the control technique, while not achieving very high removal efficiencies, may have applicability in older areas as part of an overall control plan.

One important result of these findings is the high degree of removal achieved in the on-line retention basin. The original water quality plan included sedimentation basins as part of the control strategies. It was assumed that in order to achieve a high degree of removal for TSS and T-P that chemical addition would be needed. It now appears that these removal efficiencies (>75%) can be achieved without chemical coagulation. Thus, a cost savings can be achieved in both chemical usage and especially in sludge disposal.

The alternatives of the water quality management plan will need to be updated based on the findings of the overall NURP project. Some of the original alternatives, such as street sweeping, may not prove to be as effective as originally assumed while other control strategies may be more effective than originally anticipated. The output of the nationwide program will be important to the Tri-County area given the importance of urban runoff to the quality of the river systems. It is quite apparent that sound overall control strategy for the combined sewer overflows and the other non-point sources is needed if water quality deterioration is to be stopped in this area.

Lansing Metropolitan Area Groundwater Management Concerns

The primary interest in stormwater runoff management in the Lansing Metropolitan Area (LMA) has focused around two issues. These issues include groundwater quality management and groundwater supply-stormwater recycling efforts. The interest in stormwater and groundwater management has evolved as a reaction to floods, groundwater supply and groundwater contamination problems that have occurred in the LMA. Flooding of homes, commercial areas, industries and streets, has been identified as a serious problem in the LMA and is steadily becoming worse as new development enhances runoff. Groundwater management problems have consisted of a decline in the LMA groundwater table water supply, and the contamination of a major groundwater recharge area via a poorly constructed landfill.

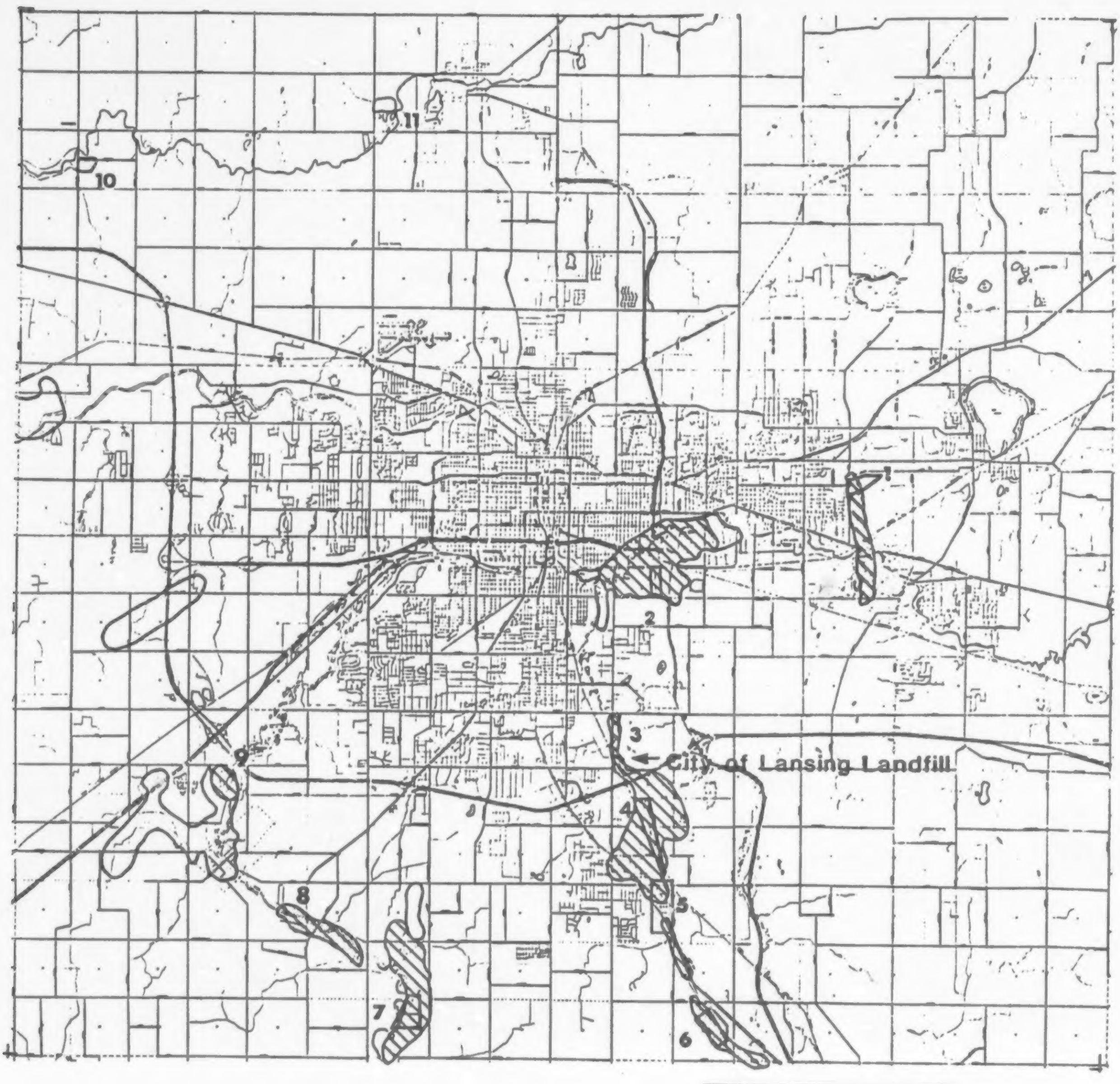
Management efforts, which address the two issues referred to above, currently underway will be summarized in the following discussion.

Groundwater Contamination Prevention

The major emphasis for preventing groundwater contamination in the LMA has consisted of the identification of areas which have been referred to as "Good" recharge areas. "Good" recharge areas are defined as areas which have predominantly sand and gravel soils overlying sandstone bedrock. The methodology utilized in the identification of recharge areas consisted of a two step procedure. The first step included the utilization of well log and soil map data to locate potential recharge areas. The second step involved utilizing electrical resistivity and soil boring procedures to verify whether or not the areas identified as potential "Good" recharge sites are actually deserving of this classification. The areas which have been studied and identified as recharge areas are identified in Figure VII-21.

Presently, work is being conducted on developing a model zoning ordinance which will determine what type of development or use will be permitted within identified "good" recharge areas. The information compiled in the Bogus Swamp Drain NURP study and other U.S. EPA NURP studies will be utilized to help assess whether stormwater runoff evolving from parking lots and streets associated with residential, commerical and industrial areas can result in groundwater contamination.

AREAS OF PROBABLE GROUNDWATER RECHARGE TO THE SAGINAW FORMATION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City of Lansing Landfill
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LEGEND

▨ Potentially Goud Recharge Area

☐ Marginal Recharge Area

▨ Areas verified as a Good Recharge Area

0 On-site investigation occured

SCALE IN MILES
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1
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3
4

TRI-COUNTY REGIONAL
PLANNING COMMISSION

Figure VII-21

Groundwater Supply-Stormwater Recycling Efforts

Stormwater is presently being considered as a recyclable resource which can be utilized in two different systems to conserve the LMA groundwater resource. The first system involves the possible use of stormwater runoff in specified areas as a source of rechargeable water. The areas which have been considered as possible sites for artificial recharge of groundwater include those identified as "Best" in Figure VII-21. The use of local stormwater runoff will be dependent upon whether the quality of the stormwater is suitable for recharge purposes.

The second groundwater recharge system is a combination flood control-irrigation water pollution control system. Increased development within the LMA has resulted in increased stormwater runoff which has enhanced flooding. In order to prevent additional flood problems, the Ingham County Drain Commission is requiring that on-site retention basins be constructed at major develoment sites. Although the retention basins are designed for flood control purposes, they can also function as water quality control and irrigation storage facilities. Recent U.S. EPA Nationwide Urban Runoff Program (NURP) studies have indicated that retention basins can reduce suspended solids, lead and phosphorus pollutant loads up to 60-70 percent. Since golf courses, major apartment complexes, and malls in the LMA often heavily irrigate lawns during peak summer demand periods, and considering that most of these establishments have on-site retention basins, the retention basins could serve a multipurpose role. Two retention basins in the Lake Lansing, Meridian Township area are located close to golf courses. The parks were located as possible alternative stormwater irrigation sites. The LMA Groundwater Management Plan has recommended that any major development proposed within the LMA, which requires an on-site retention basin, consider use of the retention basins for flood control and irrigation water reservoir purposes. Information assembled in the NURP studies indicate that small modifications on retention basin design can greatly enhance usage of these facilities as water control structures. The county drain commissioners offices will be responsible for requiring the implementation of pollution control facilities.

The authority for mandating pollution control devices in retention basins is presently granted by the Michgan Drain Code (Act No. 40, 1956). The implementation of irrigation systems will have to occur on a voluntary basis. An understanding that use of stormwater for irrigation purposes can save irrigators well construction and pumpage costs as well as reduce demand on groundwater at times when groundwater demands are seasonally high should encourage implementation of these facilities.

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SECTION VIII

EXECUTIVE SUMMARY

US EPA NATIONWIDE URBAN RUNOFF PROGRAM
LANSING, MICHIGAN

EVALUATION OF URBAN STORMWATER RUNOFF AND MANAGEMENT PRACTICES

August 26, 1983

SUBMITTED BY:

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SUMMARY

GENERAL

The study area is located in the Bogus Swamp Drain Drainage District, which is located in the Charter Township of Lansing, Ingham County, Michigan. The Drainage District is west of, and contiguous to, the City of Lansing,and representative of developed urban conditions in the City of Lansing and the larger urban region. The Drainage District is a tributary to the Grand River. The total drainage area of the Grand River at Lansing is approximately 1230 square miles. The mean flow of Grand River at Lansing is 824 cubic feet per second.

The terrain is typical Michigan glacial landscape with gently rolling topography and relatively low slope; the surface elevation at the headwaters is 890 feet and at the outlet 870 feet. The district is 450 acres and much alteration of the land surface has occurred as the district has developed with urban uses. A major part of the topographic changes were undertaken to facilitate residential, commercial and industrial development, on a subdivision or major-site basis. Table 1 indicates the land use features of the study area.

OVERVIEW, PROGRAM RESULTS

An assessment of an on-line retention basin, two in-line upsize pipe reaches, and an off-line detention basin was undertaken. The study determined the effectiveness of the above mentioned flood control treatment techniques in mitigating the pollutant impacts of urban runoff, as well as the operating costs associated with such installations.

TABLE 1
LAND USE FEATURES OF THE BOGUS SWAMP DRAINAGE DISTRICT

		Cumulative Area (area)		
Land Use		Area	Impervious Area	Effective Impervious Area
1120	Residential (.5 - 2 units/acre)	126.5	37.4	24.4
1130	Residential (2.5 - 8 units/acre)	76.9	28.2	18.5
1140	Residential (8 units/acre)	14.3	8.3	6.5
1202	Commercial (Linear strip)	13.2	9.7	9.7
1203	Commercial (Shopping center)	10.1	10.1	10.1
1301	Industrial (light)	3.3	2.2	2.2
1303	Industrial (heavy)	83.2	52.0	52.0
1400	Parkland	91.0	5.6	5.6
1401	Institutional	34.1	12.5	12.5
	Total	452.6	166	141.6

Catchment Population	2,250
Catchment Slope (ft./mi.)	221
Channel Slope (ft./mi.)	32
Length of Channel (ft.)	49,500

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Lansing, Mi
4-7-83

The on-line retention basin receives runoff from 30.3 acres of mixed residential and commercial land use plus 36.7 acres of parkland. Twenty nine events were monitored with rainfall depths ranging from 0.14 to 1.02 inches.

The two in-line upsize pipe reaches are arranged in series within the existing storm drain system. The upstream pipe received runoff from 74.9 acres of mixed land use including residential, industrial and institutional. Eighteen events were monitored at the upstream upsize pipe, with rainfall amounts ranging from 0.22 to 1.63 inches. The downstream upsize pipe receives runoff from an additional 89 acres of mixed land use types for a total of 163.9 acres. The downstream land use mix includes residential, industrial, institutional, commercial, and parkland. Seventeen events were monitored at the downstream pipe, with rainfall amounts ranging from 0.34 to 0.96 inches.

The off-line detention basin drains 127.6 acres of mixed land use types, including residential, commercial, institutional, and parkland. The basin was monitored twice during the course of the study, once during an 0.86 inch rainfall and once during a 2.82 inch rainfall.

Concentrations of solids, nutrients, and metals were routinely monitored. The data obtained were used to determine water quality benefits, if any, derived from the treatment techniques, and land use/cover effects on storm runoff. Statistical analyses were used to determine relationships of pollutant concentration and load to total rainfall. In general it was found that parameter concentration was independent on rainfall and log-normal in distribution. Pollutant load was generally dependent on rainfall amounts.

The on-line retention basin was found to be very effective in treating a wide range of rainfall amounts and pollutant loads. The in-line upsize pipes displayed variable effectiveness depending on rainfall amount and runoff volume. The off-line detention basin had little or not effect on water quality.

ON-LINE RETENTION BASIN RESULTS

General

The basin, a water hazard at a golf course, was formed from a lowland area by incresing depth and surface area through removal of natural material and contouring the basin side. The basin receives both the orginal drainage from a portion of the golf course and the re-routed flow from a storm sewer network serving upstream commercial and residential areas.

The main design goal for this basin was to provide flood relief for downstream areas by reducing peak flow rates during large storm events. A second goal was to provide control of the water quality of the runoff water discharged from the basin. This goal was incorporated into the design by providing a highly restrictive maximum outlet flow, thus increasing the retention time in the basin. The relatively large capacity available in this basin in relationship to the drainage area allowed for the extra capacity required beyond normal flood control requirements. A third objective of the design was to provide the possibility of a water supply for irrigation requirements of the golf course.

The discharge rate to a maximum of 2-3 cfs under pressure flow conditions is controlled by the outlet structure and downstream pipe. The pond outlet structure consists of a standard pipe structure made of precast concrete with a Neenah-type gate inlet. The structure is four feet in diameter and four feet deep. An eight inch diameter shear gate at the invert elevation provides the ability to lower the water elevation for maintenance or dredging purposes. A circular baffle is installed around the outlet to force a downward movement of water prior to discharge through the outlet. The baffle is present to minimize surface effects in the pond, minimize short circuiting, and to prevent discharge of floating materials during average storm events.

The restricted outflow increases detention time thereby facilitating the sedimentation process. Sedimentation was expected to be the primary method of water quality improvement.

Total Suspended Solids

Removals of total suspended solids ranged from net export to 99% removal over the course of the study. However, overall removal efficiency was usually very high, on the order of 80-90%. Mass loads and concentrations were highly variable at both the inlet and outlet of the basin. TSS inlet loadings ranged over two orders of magnitude, (from 4.3 to 460 pounds). Average inlet concentration ranged from 6 to 495 mg/1. Basin effluent loads covered three orders of magnitude (0.75 to 160 pounds), with an event mean concentration range of 2-44 mg/1. Total suspended solids concentration appears to be independent of rainfall.

Particle size distribution was examined to determine whether there existed a relationship between this parameter and removal efficiency. Regression and correlation analyses indicated no significant relationship between particle size distribution and removal efficiency. Nor was there any statistically significant relationship between particle size distribution and total rainfall. Particle size data shows a clear shift in distribution between the inlet and outlet of the basin, with the outlet containing a higher portion of smaller particles.

Total Phosphorus

Total phosphorus (TP) removal was highly variable, from net export to 94% removal. Inlet phosphorus load and concentration are independent of rainfall. Inlet concentrations ranged from 0.011 to 0.76 mg/l and are log normal in distribution, the geometric mean being 0.12 mg/l. Total phosphorus load and concentration are independent of rainfall at the outlet as well, with geometric mean concentration being 0.05 mg/l.

Total phosphorus at the inlet is primarily in the insoluble form, as shown by a correlation between TP concentration and TSS concentration. At the outlet a far greater proportion of the total phosphorus is in the soluble form. The change in composition between the inlet and outlet would suggest that phosphorus removals should be commensurate with TSS removal. The linear regression and the scatter plots both indicate that TSS removal and TP removal are linked, however, there is considerable scatter in the data.

A mass balance of inlet TP and outlet TP over the course of the study period showed a removal of 50%. As noted above, mean concentration at the inlet is 0.12 mg/l and at the outlet 0.06 mg/l.

Total Kjeldahl Nitrogen

Total Kjeldahl nitrogen (TKN) removal efficiency displays the same type of variablility as TP. TKN was exported from the basin during 10 of the 29 monitored events, while, during one event 98% of the influent mass was removed. Inlet concentration of TKN is independent of rainfall and appears log normal in distribution. Outlet concentration was neither clearly log normal nor Gaussian in distribution.

A mass balance of TKN input and output over the study period indicates 18 percent of the influent mass was removed by the basin. Using median concentration values, removal efficiency was observed to be 28 percent.

Total Kjeldahl nitrogen concentration at the basin inlet displayed correlation with TSS concentration, while at the outlet there was very little correlation. This association of TKN with the particulate matter is confirmed by the percentage of soluable Kjeldahl nitrogen at the two stations, with the inlet exhibiting a much higher percentage of insoluable Kjeldahl nitrogen than at the outlet.

Biochemical Oxygen Demand

Biochemical oxygen demand (BOD) was removed by the retention basin during 13 of 18 events. The largest observed removal of BOD was 87% during one event, while removals were greater than 70% for 7 of the 13 events. Based on a study period mass balance, 44 percent of the total influent mass was removed by the basin. Regression analysis of concentration and load indicated no statistically significant relationships to rainfall at either station.

There appears to be a relationship between BOD and TSS at the basin inlet, but no corresponding relationship at the outlet.

Chemical Oxygen Demand

Chemical Oxygen Demand (COD) removal varied widely during the study period. Net export occurred during 5 of 23 events. During events when COD was removed, the removal efficiency ranged from 10 to 85%. As with the parameters mentioned previously, there is no statistically significant relationship of load or concentration to rainfall at either station.

Concentration at the basin inlet displays a log normal distribution, with geometric mean concentration equal to 49 mg/1 and a range of 6 to 160 mg/1. Outlet concentration is not accurately described by either a log normal or Gaussian distribution. In both distributions the average concentration is equal to 30 mg/1 and the median value is 32 mg/1. Concentration ranges from 14 to 44 mg/1 at the outlet.

A mass balance performed over the length of the study period shows a removal of 30% of the total mass input. Using geometric mean concentration values, removal efficiency was observed to be 38%.

Lead

Significant amounts of lead were consistently removed by the retention basin. More than 90% of influent lead was removed during 9 of 19 events, and was never exported. As is true with the other parameters in this study, concentration appears to be independent of rainfall at both the inlet and outlet. Inlet concentrations displays a log normal distribution with the geometric mean concentration equal to 0.09 mg/1. The concentration distribution at the outlet is not adequately discribed by either log normal or Gaussian function, although a log normal distribution is assumed. The geometric mean concentration at the outlet is equal to 0.0006.

Based on a mass balance for all monitored events, the retention basin removed 90% of the influent lead load. Using the geometric mean concentration for inlet and outlet the removal efficiency was 93%.

Lead in stormwater is generally adsorbed on particulate matter. Therefore, it could be expected that lead removal would correlate well with suspended solids removal. A linear regression of lead removal versus TSS removal indicated that the two are not related at a high degree of correlation. On a percentage basis, lead removal is consistently higher than TSS removal. While there is weak correlation between TSS removal and lead removal, there is a correlation between concentration of suspended solids and lead concentration at the basin inlet. However, there is no similar correlation at the outlet. This would suggest that influent lead is mostly insoluble and adsorbed on particulates and is being flaculated and settled in the basin. In the basin water column there is a low level of background lead, measured at 0.0003 mg/l during one dry weather survey. Background lead plus that adsorbed on fine particulates is what is discharged from the basin.

Copper

Copper removals are variable than those shown by lead, ranging from net export to 94% removal. The basin removed copper during 12 of 14 events. Copper concentration and load are independent of rainfall at both stations. Concentration at both inlet and outlet are best described by a log normal distribution with mean concentration at the inlet equal to 0.01 mg/l and at the outlet, 0.005 mg/l.

A study period mass balance indicates 36% of influent mass was removed by the basin. Removal efficiency based on the geometric mean concentration was 50%.

Snowmelt

The concentrations for all parameters, with the exception of influent TDS and chloride, fell well within the range observed during rainfall events. Influent TDS and chloride were both an order of magnitude greater than that observed during rain events. Approximately half of the TDS can be accounted for by the chloride contributed by road salt.

On a mass balance basis, the basin removed 66% of the influent TSS load and lowered the concentration from 110 mg/1 to 17 mg/1. Eighty percent of the influent lead load was removed while lead concentration declined from 0.23 mg/1 to 0.02 mg/1. The difference between load removal and removal based on concentration is due to the flow imbalance at the two stations. Similiar concentration reductions were noted for other parameters, especially, the metals. Nutrients showed little or no reductions in most cases, however, it is of interest that nutrients at the inlet usually had a greater fraction in the insoluble form than at the outlet. Nutrients at the outlet were almost completely in the soluble form (e.g., TP-100%, TKN-91%, TOC-100%.

Conclusions

It was anticipated that removal of pollutant materials in the basin would be accomplished primarily by sedimentation of suspended solids. The principal factor affecting sedimentation under ideal conditions is the detention time. Under such conditions detention time will determine the critical settling velocity and all particles having a settling velocity greater than the critical velocity will be removed. Actual performance of the basin will depend on the magnitude of deviation from ideal conditions due to short circuiting, turbulence, and scour.

The basin has a restricted outflow structure that limits effluent flow to 2-3 cfs under pressure conditions. The structure attenuates the flow and increases detention time to allow sedimentation of storm generated solids. The restricted outflow also decreases the potential for short circuiting, scour and turbulence during peak flow periods.

Detention time, while important, is not the only consideration in determining removal of suspended solids. Events with the longest detention time do not necessarily result in the greatest removal percentage. Inlet concentration and particle size distribution play an important role in the removal percentage obtained.

The dry weather data suggests that there is a particle size fraction that does not settle regardless of detention time and also that there is a certain background concentration in the basin.

In the case of storms with a low influent TSS concentration or storms which have a small percentage of settleable particle, the background concentration can become a significant portion of the effluent load and directly affect the percent removal calculation.

It would appear from the data collected that the effluent concentration from the basin is not dependent on the storm size, detention time, inlet concentration or other physical factors which normally would affect the sedimentation processes. The discrete events sampled showed that the concentration at peak flow periods was within the range of all of the events monitored. Thus, even high peak loading rates appear to have little affect on the effluent concentration.

Given the independence of the effluent concentration on all factors tested, the basin is apparently achieving the maximum level of control which can be attained within reasonable basin size by the sedimentation process. Indeed, the basin may actually be over-sized for the drainage area supplying runoff to the basin. The data which was collected does not allow for an evaluation of the impact that a smaller basin with higher outlet flows would have on removal levels. It is possible that the basin is designed at the edge of the size-removal relationship or that a significantly smaller basin could be utilized to achieve similar removals.

Effluent from the basin is generally as good as or better than tertiary treatment standards. While this comparison does not indicate whether or not this effluent water quality will have a negative impact on a receiving stream, it does show that the basin effluent is comparable with the strict discharge levels currently being set for point source discharges.

Costs

Cost data for the On-Line Retention Basin is presented in Table 2. The costs of the inlet and outlet pipes include excavation, placing the pipe, backfilling with the appropriate material and compacting, fertilizing and reseeding the damaged area, and hauling the excess excavated material to a dump site one mile away. These costs were based on cost factors found in the 1980 Dodge Guide. The costs for the manholes were taken from the contractors bid documents.

TABLE 2

Cost Data for The On-Line Retention Basin

Capital Cost (in 1980 Dollars)

Inlet Pipe and Manholes	$ 23,500.00
Outlet Pipe and Manholes	19,300.00
Retention Basin	62,400.00
	$105,200.00
Engineering Design and Inspection	$ 11,800.00
Review and Approval of Design Plans	1,200.00
Legal and Financial Fees	1,050.00
Contingency	7,150.00
	$ 21,200.00
Total Installed Capital Cost	$126,400.00

Annual Operating Cost (in 1980 Dollars)

Routine Maintenance	$ 600.00
Waste Disposal	350.00
Capital Cost*	6,320.00
Interest**	7,510.00
Total Annual Operating Cost	$ 14,780.00

*Straight-line depreciation of total installed capital costs over 20 years.
**Bonds at 5.9419%, principal of $126,400.00.

TCRPC/NURP
Lansing,Mi
4-7-83

IN-LINE RETENTION BASIN RESULTS

General

The in-series upsize pipes are designed to serve as sediment traps by slowing the water velocity, providing detention time, and allowing the sedimentation process to occur. The pipes provide a sump for deposition of settleable materials, which can then be removed by periodic cleaning.

A wide range of rainfall amounts and flows were monitored during the study period. The smallest rain monitored at any of the upsize pipe stations was 0.14 inches, while the largest was 1.63 inches. Runoff volumes varied from 3,000 to 135,000 cubic feet.

As noted previously, the two upsize pipes drain an area of mixed land uses. The upstream pipe drains 75 acres, with a 31% effective imperviousness. The downstream pipe drains another 89 acres (13% effective impervious), for a total of 164 acres of which 21% is effectively impervious. Runoff, as a percentage of rainfall, is approximately the same for both units. The mean runoff coefficient for both upsize pipes is 0.12.

It is anticipated that water quality improvement in the upsize pipes would occur primarily through settling of suspended solids. Treatable nutrients and metals will generally be insoluble and/or associated with settleable solids. Therefore, it can be expected that overall water quality improvements will be related to the sedimentation process.

Total Suspended Solids

The upstream upsize pipe displayed highly variable removal efficiences during monitored events. Affect on suspended solids ranged from net export of material during three events to a high of 66% removal during the 14 events when removal occurred. Typically, suspended solids removals were in the 30%-40% range. A mass balance calculated over the study period for all monitored events showed the upstream pipe removed 12% of the measured influent mass of 4,000 pounds of suspended solids.

There was no statistically significant linear relationship between TSS concentration and rainfall at the upstream pipe. A linear regression analysis of suspended solids load indicated a relatively high correlation to rainfall at the upstream pipe. Removal of suspended solids is related to the particle size distribution and detention time. The data indicates a shift in particle size distribution between the inlet and outlet of the upstream pipe. It should also be noted that a relatively small percentage of the total is in the greater than 62.5 micron range.

The downstream upsize pipe displayed relatively low removals on a straight percentage basis (net export to a maximum of 36% removal). Net export occurred during 4 of 16 events. A mass balance for all events over the study period indicated no removal of the suspended solids input of 7,000 pounds by the downstream pipe.

The particle size distribution at the downstream unit does not show the shift in size between the inlet and outlet as was observed for the upstream pipe. However, the low percentage of particles in the greater than 62.5 micron range is similar.

Total Phosphorus

On an event basis, TP removal was highly variable at both upsize pipes. Treatment efficiency ranged from net export to 40% removal at both pipes. A mass balance for all events monitored during the study indicated no overall removal of TP by either pipe. The total influent loads were 7.5 pounds and 15 pounds for the upstream and downstream pipe, respectively.

Total Kjeldahl Nitrogen

Total Kjeldahl nitrogen (TKN) displays the same type of variability as TP. Event removal of TKN ranged from net export to 30% removal at both pipes. A study period mass balance showed no removal of an influent load of 30 pounds at the upstream pipe and no removal of 70 pounds at the downstream pipe.

Biochemical Oxygen Demand

Biochemical oxygen demand (BOD) typically showed little or no removal on an event basis. Export of BOD was frequent although removal of up to 30 to 40 percent of influent mass was observed. A study period mass balance indicated no removal of BOD at either pipe. BOD input over the study period to the upstream pipe was 60 pounds and to the downstream pipe was 195 pounds.

Chemical Oxygen Demand(COD)

Event removal was generally very low. Export occurred frequently and maximum observed removal was 40%. A mass balance of all monitored events indicated no removal of COD at either upsize pipe. Influent mass input to the upstream and downstream pipes was 600 pounds and 1,800 pounds, respectively, over the duration of the study.

Lead

As with the other parameters discussed above, lead removal is highly variable. However, lead is removed more consistently than the nutrients and is not frequently exported. The study period mass balance indicated that the upstream pipe removed 25% of an influent mass of 1.3 pounds and the downstream pipe removed 50% of an in mass of 6.4 pounds.

Copper

Copper removal displayed very little consistency on an event basis. Typically, when removal occurred it was low, (in the 10-20% range) although removals in the 90% range were observed in the upstream pipe. A study period mass balance indicated that the upstream pipe removed 60% of the influent mass of 0.3 pounds while the downstream pipe removed none of influent mass of 0.7 pounds.

Accumulated Sediment

As part of the program for evaluating the performance of the upsize pipe system, the pipes were emptied of water and the amount of sediment measured and sediment samples collected.

The measured volume on 4/14/82 was 170 cubic feet in the upstream pipe and 65 cubic feet in the downstream pipe.

Both upsize pipes were cleaned in early June, 1981. A combined total of 380-430 cubic feet (wet volume) was removed. Therefore, from June to November, 1981 the upstream pipe accumulated approximately 4,200 pounds of sediment and the downstream pipe collected approximately 5,000 pounds.

Conclusions

A mass balance including all monitored events indicated little or no removal of pollutants at either upsize pipe. However, visual inspection of the pipe sumps and the cleaning of the pipes indicated substantial amounts of sediment had accumulated.

During the five month period after the pipes were cleaned, the upstream pipe accumulated approximately 4,200 pounds of sediment. During that period 20 precipitation events with a depth greater than 0.1 inches occurred, seven of which were monitored. Eight of the 20 storms were less than 0.25 inches. During the total study period 20 events were monitored at the inlet with a mean depth of 0.59 inches and a range between 0.17 inch and 1.63 inch. Total measured input load during the study period was 4,000 pounds.

Storms with less than 0.3 inches typically had a suspended solids load from 50-150 pounds. Storms from 0.3 to 1 inch typically had suspended solids loads of 100-500 pounds. Therefore, assuming eight storms with an average load of 100 pounds and 12 storms with 300 a pound load, the total inlet load would have been approximately, 4,400 pounds. Hence, to account for the amount of solids in the pipe - using observed mass load rates - average removal efficiency would have to be close to 100%.

A possible explanation for the discrepancy in removal efficiency predicted by the monitoring program and the accumulated material observed in the pipe is that the sampling protocol was missing the larger sand-like materials. Visual inspections of the pipe sump revealed a large pile of sandy material near the inlet of the pipe. It was found that the pile of sand comprised 40-50% of the total mass in the pipe. The remaining material was made up of finer, clay-organic type material. Sand type material usually rolls along the pipe invert and constitutes the bed load of the drainage system.

The Quality Assurance (QA) data indicates that the sampling equipment was effective in sampling the area around the screen. However, QA grab samples would probably not be any more effective in sampling bed load than the suction lift samplers.

A mass balance over the entire study period for both units show little or no removal of the various nutrients and metals, with some exceptions. However, on an event basis nutrients and metals were removed by the upsize pipe system, although as noted in the discussion on removal efficiency, these removals were highly variable.

The data suggests that small events tend to display good removal of nutrients and metals while larger events tend toward lower removals of these materials. Small storms and large storms of long duration result in low velocities and long detention time. Low velocity trends to minimize scour and long detention time facilititates removal of small particle sizes. However, removal and export is so variable that no statistically significant trends could be identified -large high intensity storms occasionally displayed relatively high removals of pollutants while small events periodically resulted in export of nearly every parameter method.

Analysis of accumulated sediment clearly indicates that removal of nutrients and metals is occurring, even though a mass balance of runoff samples for all events indicate that little or no removal is taking place. However, the mass balance removal efficiency is misleading, because typically only the larger storms generated enough flow to monitor.

Rainfall data collected during the study period show numerous storms in the 0.1 - 0.3 inch range. These small storms contribute to the overall accumulation of solids, however, do not allow any estimation of load or removal efficiency during these events, and therefore the amount contributed by the unmonitored events is unknown although that unmonitored bed load of heavier inert material contributes a large portion of the accumulated solids.

The principal factor under ideal conditions which affects sedimentation efficiency is detention time. In the upsize pipe system detention time varies with velocity and is a function of total rainfall and intensity. Average detention times ranged from 10 to 120 minutes in the upstream pipe and from 9 to 70 minutes in the downstream pipe. Minimum detention time will be signficantly less than average detention time during peak storm intensity. It should also be noted that minimum detention time will occur when the greatest load of material is expected to be entering the system.

The effect of velocity on removal efficiency of TSS was examined. The data indicated that there is no statistically significant relationship between removal of TSS and velocity at either upsize pipe.

Costs

The cost data for the in-line upsized pipes are included in Table 3. The cost includes excavation, placing the pipe, backfilling with the appropriate materials and compacting, and hauling the excess excavated material to a dump site one mile away, in addition to the cost of the pipes themselves. These costs were based on cost factors found in the 1980 Dodge Guide. The costs for the manholes and pavement replacement were taken from the contractors bid documents.

TABLE 3

Cost Data of In-Line Retention Basin

Capital Cost (1980 Dollars)

	Upstream Pipe (144'L, 96"D)	Downstream Pipe (85'L, 96"D)
Upsized Pipe	$ 48,900.00	$ 30,100.00
Manholes	6,000.00	6,000.00
Replace Pavement	3,100.00	1,900.00
Subtotal	$ 58,000.00	$ 38,000.00
Engineering Design and Inspection	$ 6,500.00	$ 4,300.00
Review and Approval of Design Plans	1,200.00	1,200.00
Legal & Financial Fees	600.00	400.00
Contingency	3,900.00	2,600.00
Subtotal	$ 12,200.00	$ 8,500.00
Total Installed Cost	$ 70,200.00	$ 46,500.00

Annual Operation Cost (1980 Dollars)

Routine Maintenance	$ 240.00	$ 240.00
Waste Disposal	2,550.00	1,500.00
Capital Cost*	3,510.00	2,325.00
Interest	4,171.00**	2,763.00***
Total Annual Operating Cost	$ 10,471.00	$ 6,828.00

*Straight-line depreciation of total installed capital cost over 20 years.

**Bonds at 5.9419%, principal of $70,200.00

***Bonds at 5.9419%, principal of $46,500.00

OFF-LINE DETENTION BASIN RESULTS

General

The off-line detention basin was designed to provide flood relief for downstream areas during storms larger than the one year storm. According to design specifications, the basin would operate when storm generated flow exceeded the discharge capacity of the downstream pipe. The water would then flow out of a beehive opening and into an undeveloped grassy area. As a pollution control device it was anticipated the basin would act as a settling basin whenever the area was flooded.

After initiation of the flow monitoring program it became apparent that the basin was not receiving sufficient flow to flood the basin with any reasonable frequency. Therefore, an inflatable pillow was installed in the outlet to provide further restriction of outlet flow. The pillow was inflated when storm events of greater than .5 inches were forecast.

The basin was monitored twice during the study period. The first monitored event was a 2.82 inch rain (27 hour duration) that began on May 10, 1981. The basin treated 120,000 cubic feet of water or approximately 20% of the runoff water passing through the drain. The maximum detention for the water treated during the event was approximately 25 hours and the minimum approximately 15 hours. Maximum pond depth was 2-3 feet.

The second event occurred on August 27, 1981, when flow weighted composite samples were taken during an 0.86 inch rain (10 hour duration). Approximately 34,000 cubic feet of water was treated, with an average detention time of 5 hours. The basin treated about 65% of the total flow.

Conclusions

The off-line detention basin had very little effect on pollutant levels. The parameters of interest show little or no change in concentration between inlet and outlet.

The overall inlet trend toward low total suspended solids concentration and small size particle distributiuon is due to the fact that the off-line detention basin is receiving a significant portion of its influent from the on-line retention basin which is located upstream and thus has already been treated to the maximum attainable level by the sedimentation process alone.

Another factor possibly contributing to the poor performance of the basin is detention time. During the August event, when approximately 65% of the flow was detained, average detention time was 5 hours. Based on ideal settling, particles with a diameter of 10 microns have a settling velocity of 0.8 ft./hr. in quiescent conditions. Using a pond depth of 2 feet, particles with a diameter greater than or equal to 10 microns should settle in 2.5 hours. Obviously, however, conditions in the basin are far from ideal, with wind effects, turbulence, and the like making the required detention time for settlement of such particles much greater. Once the particle has settled it must adhere to something to be removed from the water column. However, when the basin empties, water exits from the bottom. As the water drains out, velocities near the basin floor may approach scour velocity for these smaller particles, and they could become resuspended. In fact, if water velocity is high enough, some erosion could take place and the basin could actually contribute suspended solids. Some erosion effects were noted in the vicinity of the beehive inlet/outlet structure.

Given that the particle size distribution of the influent water tended toward the low end in all samples, the detention time in the basin was probably not long enough to provide any high degree of settling during the August event (5 hours average).

The site specific conditions of low suspended solids, small particles, and bypass flow found in this study resulted in a set of circumstances which make it difficult to determine if dry detention basins of this type could ever achieve significant control of pollutant loads. It is possible that if dry basins receive higher loadings of pollutants, particularly with larger suspended particle sizes, that some sedimentation would be achieved. However, observation of the off-line basin indicates that the problems of turbulence near the sediment-water interface during the draining of the basin and erosion near the outlet may result in poor performance due to resuspension of the settled particles.

This observation is further strengthened by visual inspection of several dry on-line basins in the southern Michigan area. In each case muddy bottom conditions and signs of erosion or channelization through these on-line basins is quite obvious. It is probable that with this type of condition occurring in these basins, performance for the control of pollutants is marginal to poor. It is also possible that these types of problems could be overcome by special design considerations during the planning phase. However, based on our observations, cooperation of the owners or inspection by the authorized public agency will be needed to insure that the appropriate design measures are utilized and installed properly. One noticeable problem in most dry basins is obtaining good sodding and/or bank stabilization at the end of construction. Without proper finishing details such as these, the dry basins exhibit severe erosion problems, particularly during the spring periods which characterized by high flows and less well established plant growth.

Costs

Cost data for the Off-Line Detention Basin is presented in Table 4. Since no excavation or contouring of the existing basin was required the only improvement needed for this construction of the BMP was the installation of the pipe from the storm sewer to the basin. The cost of the pipe includes excavation, placing the pipe, backfilling with the appropriate material and compacting, fertilizing and reseeding the damaged area, and hauling the excess excavated material to a dump site one mile away. These costs were based on cost factors found in the 1980 Dodge Guide. The cost for the manhole was taken from the contractors bid documents.

TABLE 4

Cost Data For The Off-Line Detention Basin

Capital Cost (in 1980 Dollars)

Inlet Pipe	$ 5,950.00
Manhole	800.00
	$ 6,750.00
Engineering Design & Inspection	$ 760.00
Review & Approval of Design Plans	600.00
Legal & Financial Fees	70.00
Contingency	1,890.00
Total Installed Capital Cost	$ 8,640.00

Annual Operating Cost (in 1980 Dollars)

Route Maintenance	$ 240.00
Capital Cost*	432.00
Interest**	$ 513.00
Total Annual Operating Cost	$ 1,185.00

*Straight-line depreciation of total installed capital cost over 20 years.
**Bonds at 5.9419% - principal of $8,640.00.

LAND USE EFFECTS ON POLLUTANT LOADS

General

Two evaluations were used. The were land use versus event geometric mean concentration (EMC) and land use versus load.

Land Use Versus EMC

Using seven selected parameters (TSS, TP, TKN, BOD, COD, Pb, and Cu), the district's sampling sub-basins were ranked.

The area with the highest overall concentrations is characterized by a large amount of older residential area with predominatly ditch swale drainage. The area has a large unpaved parking lot and school yard. Area driveways are often dirt or gravel. Visual inspection of the area also indicates some erosion in the ditch and swale drainage system.

Normally it would be anticipated that commercial and industrial areas would display higher concentrations than a residential area. In fact, the industrial area (Station Two) occupies the second ranking and displays the highest concentration of TP, COD, and copper; however, TSS and TKN values are lower. Station Six, which possesses a large amount of commercial area (shopping center) has the lowest overall concentration ranking. The low concentration values may be somewhat unexpected considering the relatively large amount of commercial and high density residential uses in this area. The low concentrations may be a result of a dilution effect due to the large amounts of runoff generated by the area.

Land Use Versus Load

Highly urbanized areas would be expected to deliver larger loads on a pound per acre basis, given comparable event mean concentrations. The loads were calculated using average annual rainfall, mean runoff coefficient, and geometric mean EMC for each station.

Station Six (commercial area shopping center) displays the highest loading on a pound per acre basis. It occupies this position due to the large amount of runoff generated in the area. The other areas, which have similar runoff coefficients, maintain the same relative rankings displayed in the EMC ranks. The Station Two (industrial parking lot) estimated load would have the highest overall ranking. These results show that loads generated in the Bogus Swamp Drain district are primarily a result of differences in flow. The concentration values are all fairly close, with some exceptions, and indicate there is not a great deal of difference in runoff water quality among the various land use mixes.

On a load basis the land use type exhibiting the heaviest pollution is the industrial (assuming no diversion of runoff), followed by commercial/residentail, and residential areas.

Conclusion

The significant difference in pollutant loads (pounds/acre) between commercial/industrial and residential areas was found to be primarily due to the increased volume of runoff associated with the commercial/industrial areas. Recognition of this factor is important when considering the type, and particularly the size, of various control alternatives.

RECEIVING WATER EVALUATION

General

The Grand River is designated by the State for the following uses: agricultural, navigation, industrial water supply, warm water fish, and partial body contact. A portion of the Grand River upstream from the Bogus Swamp discharge is designated for total body contact recreation. There are no State numerical standards for some parameters including total suspended solids, phosphorus,and nitrogen. Basically, there is no comprehensive list of numerical standards to compare with storm water quality data. However, certain guidelines were assumed for means of comparison and include: suspended solids - 25 mg/l, total phosphorus - 0.5 mg/l, ammonia nitrogen - 0.4 mg/l, and total organic nitrogen - 0.8 mg/l.

Violations

Based on historical and program sampling data, the average concentration of almost every pollutant parameter (with the possible exception of the metals) exhibits a significant increase in the Lansing urban area. Standards and guidlines were frequently exceeded for total suspended solids, total dissolved solids, and dissolved oxygen and occassionally exceeded for phosphorus, ammonia nitrogen, and total organic nitrogen.

The violations are caused by the factor that in this specific area there is a large population contributing relatively large sewage and wet weather runoff flows to comparably small rivers. In the case of dry weather conditions, the wastewater treatment plants can account for 25 - 50% of the total river flow. During wet weather conditions, urban stormwater runoff (CSO and separate) can frequently double or triple the river flow. This condition clearly shows that if the principal sources of flow to the river exceed the water guidelines or standards for the river, then violations will occur in the river.

Conclusions

The results of this study show that the on-line retention basin is capable of controlling the two pollutants, suspended solids and phosphorus, to a high degree. This control strategy is applicable to the problems associated with the separate storm systems and this river system. In addition, this technology is readily applicable to developing areas which will only add to the existing water quality problems if they are permitted to develop without controls. It also appears that, given the high degree of control achieved by this type of basin, some urban runoff could be allowed to be uncontrolled and blended with the effluent of the retention basin to achieve the overall desired objective. This provides for a degree of flexibility in implementing control alternatives.

The upsized pipes were much less effective in controlling suspended solids and phosphorus. However, the longer pipe did show removals in the range of 25% (concentration basis). A somewhat larger pipe, with an improved design to minimize scour, could possible achieve 25-35% removal on a routine basis. These removals are in the same range as those required in the overall regional area. Thus, this control technique, while not achieving very high removal efficiencies, may have applicability in older areas as part of an overall control plan.

One important result of these findings is the high degree of removal achieved in the on-line retention basin. The original water quality plan included sedimentation basins as part of the control strategies. It was assumed that in order to achieve a high degree of removal for TSS and TP that chemical addition would be needed. It now appears that these removal efficiencies (>75%) can be achieved without chemical coagulation. Thus, a cost savings can be achieved in both chemical usage and especially in sludge disposal.

SECTION IX
FINAL REVISIONS AND ADDITIONS FOR USEPA NATIONWIDE URBAN RUNOFF PROGRAM
LANSING, MICHIGAN

EVALUATION OF URBAN STORMWATER RUNOFF
AND
MANAGEMENT PRACTICES FOR CONTROLLING URBAN STORMWATER RUNOFF

August 26, 1983

Submitted by:
Tri-County Regional Planning Commission
913 W. Holmes, Suite 201
Lansing, Michigan 48910
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This report was funded in part by a grant under PL 92-500 as amended by the U.S. Environmental Protection Agency's Nationwide Urban Runoff Program, Chicago Region V and Headquarters, Washington, D.C.

Background

The purpose of this report is to revise the Draft Final Report, US EPA Nationwide Urban Runoff Program, Lansing, Michigan, dated August, 1982. The revisions are based on U.S. Environmental Protection Agency comments and editing by Tri-County Regional Planning Commission.

Because of the length of the Final Report, Appendices, and Data Base documents, extensive editing was not undertaken to correct all typing or english errors. Thus, the corrections noted in this report are not meant to be all inclusive.

An Executive Summary of the Final Report is available under separate cover to assist the reader in consolidating discussions, evaluations, and conclusions.

Data Base

The extensive data base is best accessed through the computerized NURP Data Management System. This contained data base is complete and corrected. There may be errors within the paper document Data Base itself. As corrected sheets (if any) cannot be obtained from the technical consultant, the Environmental Control Technology Corporation, the reader is urged to rely solely on the corrected computer version for the raw data.

Revisions

The following are final revisions in page order to the Draft Final Report, U.S. EPA Nationwide Urban Runoff Program, Lansing, Michigan. The revisions are (underlining represents new language and slants represents deletions):

416

1. Cover Page
   Add paragraph at bottom of page
   "This report was funded by grants under PL-92500 as amended by the U.S. Environmental Protection Agency, Nationwide Urban Runoff Program, Chicago Region V and Headquarters, Washington, D.C. Grant match was provided by the Bogus Swamp Drain Drainage District, Ingham County Drain Commission."

2. Table of Contents
   Add to end
   "VIII. Executive Summary VIII-1
   IX. Final Revisions and Additions ... IX-1"

3. Page II-4, first paragraph, third sentence
   Add to read
   "This study was designed to determine the efficiency of these flood control practices in the enhancement ..."

4. Page III-15, second sentence
   Change to read
   "As can be seen by referring to Table III-2, 65% of the area ..."

5. Page III-23, under Characteristics and Locations
   Spelling
   "... nearly Adrian ..."

6. Page III-30, last sentence
   Spelling
   "... separate from the sanitary lines"

7. Page III-76, third sentence
   Spelling
   "The basin was formed ..."

8. Page III-76, end of page
Add new last sentence
"As of 1984, irrigation has not yet occurred. This option is still considered viable with the golf course management".

9. Page III-81, last paragraph, second sentence
Spelling
"... through the use of the sedimentation process".

10. Page IV-24, next to last sentence
Spelling
"Approximately six samples ..."

11. Page IV-35, second paragraph, third sentence
Change to read
"Indications from US EPA Headquarters were that the pollutant data ..."

12. Page IV-39, first paragraph, fifth sentence
Add to read
"The full document has been filed with, and is available for reviewing and reference at, the Tri-County Regional Planning Commission, Ingham County Drain Commission, US EPA Region V, and EPA Headquarters and will be available for purchase at the National Technical Information Service (NTIS)."

13. Page V-15, second paragraph, second and third sentences
Add to read
"The data indicates that influent concentration is log normally distributed (Appendix IV, Statistical Analyses) with geometric mean EMC equal to 53 mg/l (Table V-1). Outlet concentration is not well described by either log normal or Gaussian districtuion (Appendix IV, Statistical Analyses); ...

14. Like No. 13 above, similar discussions occur throughout the report as to influent or outlet concentrations being log normal and/or Gaussian described. In all causes (Appendix IV, Statistical Analysis shall be the basis for any conclusions reached. These discussions occur on pages V-15, V-17, V-21, V-23, V-24, V-25, V-26, V-53, V-56, V-57, V-58, V-59, V-60, VI-35, VI-39, and VI-41.

15. Page V-32, last paragraph, first sentence
Add to read
"... data collected over the two year period ..."

16. Page V-33, second paragraph, first sentence
Spelling
"... provides the basis for comparing ..."

17. Page V-36, last line
Delete to read
"**Bonds at 5.9419%, ..."

18. Page V-58, second paragraph, second sentence
Spelling
"... relatively small sample size ..."

19. Page V-65, last word
Spelling
"... every parameter monitored"

20. Page V-71, last sentence
Spelling
"... lengths of pipe were inserted under

21. Page V-85, second paragraph, fifth sentence
Change to read
"... design measures are installed and utilized properly."

22. Page V-87, last line
Delete to read
"** Bonds at 5.9419% ..."

23. Page VII-28
Spelling, first paragraph, third sentence
"The existing data on CSO's in the study area ..."

24. Page VII-90, first paragraph, third sentence
Change to read
"... possible sites for artificial recharge of groundwater include those verified as "Good" in Figure VII-21."